(2)

US Army Corps
of Engineers
Construction Engineering
Research Laboratory

USACERL TECHNICAL REPORT N-90/14
August 1990
Technology for Renovating Army Potable Water Infrastructure

A Computer Program for Optimal Control of Water Supply Pump Stations: Development and Testing

by
Donald V. Chase

This report presents an optimal control methodology designed to reduce the electrical consumption and operating costs at Army water supply pumping stations. The methodology, contained in a computer program called Optimal Control of Pumping Stations (OCOPS), enables water utility managers and operators to reduce the electrical consumption at pumping stations while maintaining adequate storage and system pressures for fire protection. OCOPS can be applied to any water distribution system and is not limited to any number of pumps, pump stations, or storage tanks within the system. The optimal control strategy developed is based on electricity unit charges, but can be modified to include electrical demand charges. OCOPS was applied to the Fort Hood, TX, water distribution system to demonstrate the effectiveness and capabilities of the model. OCOPS calculated optimal pumping costs nearly identical to actual costs (within 3 percent), indicating that Fort Hood is already operating at the optimal level.

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY (Leave Blank)

2. REPORT DATE
August 1990

3. REPORT TYPE AND DATES COVERED
Final

4. TITLE AND SUBTITLE
A Computer Program for Optimal Control of Water Supply Pump Stations: Development and Testing

5. FUNDING NUMBERS
PR A896
WU NN-TN9

6. AUTHOR(S)
Donald V. Chase

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Construction Engineering Research Laboratory (USACERL)
2902 Newmark Drive, PO Box 4005
Champaign, IL 61824-4005

8. PERFORMING ORGANIZATION REPORT NUMBER
TR N-90/14

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
Directorate of Military Programs
ATTN: CEMP-EB
20 Massachusetts Avenue, NW.
Washington, DC 20314

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Copies are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT (Maximum 200 words)

This report presents an optimal control methodology designed to reduce the electrical consumption and operating costs at Army water supply pumping stations. The methodology, contained in a computer program called Optimal Control of Pumping Stations (OCOPS), enables water utility managers and operators to reduce the electrical consumption at pumping stations while maintaining adequate storage and system pressures for fire protection. OCOPS can be applied to any water distribution system and is not limited to any number of pumps, pump stations, or storage tanks within the system. The optimal control strategy developed is based on electricity unit charges, but can be modified to include electrical demand charges. OCOPS was applied to the Fort Hood, TX, water distribution system to demonstrate the effectiveness and capabilities of the model. OCOPS calculated optimal pumping costs nearly identical to actual costs (within 3 percent), indicating that Fort Hood is already operating at the optimal level.

14. SUBJECT TERMS
OCOPS
Optimal Control of Pumping Stations
water supply
pumping stations
costs

15. NUMBER OF PAGES
99

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
Unclassified	Unclassified	Unclassified	SAR

NSN 7540-01-280-5500

Standard Form 298 (Rev. 2-89)
Prescribed by ANSI Std 239-18
298-102

FOREWORD

This work was conducted by the U.S. Army Construction Engineering Research Laboratory (USACERL) for the Directorate of Military Programs, Headquarters, U.S. Army Corps of Engineers (HQUSACE), under Project 4A162720A896, "Base Facility Environmental Quality"; Work Unit NN-TN9, "Technology for Renovating Army Potable Water Infrastructure." The HQUSACE Technical Monitor was R. Ross, CEMP-EB.

The project was performed by Donald V. Chase, U.S. Army Waterways Experiment Station (WES), under contract to USACERL. Wayne W. Sharp, also of WES, assisted in the field data collection. USACERL Principal Investigators were Drs. Stephen Maloney and Prakash Temkar, USACERL Environmental Division (EN). Dr. R. K. Jain is Chief of USACERL-EN.

COL Everett R. Thomas is Commander and Director of USACERL and Dr. L.R. Shaffer is Technical Director.

CONTENTS

FIGURES

FIGURES (Cont'd)

TABLES

A COMPUTER PROGRAM FOR OPTIMAL CONTROL OF WATER SUPPLY PUMP STATIONS: DEVELOPMENT AND TESTING

1 INTRODUCTION

Background

Energy conservation in the United States has been an important issue ever since the oil embargo of 1973 when the public became very much aware of the unstable and limited availability of energy-producing resources. Shortly after the oil embargo, the attention of the news media, political leaders, and the American public was focused on energy-related topics which, at the time, was rare--especially for technical issues.[1] Even though the procurement of energy-producing resources has stabilized somewhat, supplies are still limited and the cost of these resources continues to rise.

Water utilities are major consumers of electrical energy. In fact, up to 7 percent of the electricity consumed annually in the United States is used by the municipal water works industry.[2] Pumping of water comprises the major fraction of electrical usage at water utilities, accounting for as much as 80 to 95 percent of the energy requirement in some systems.[3] Furthermore, energy costs are the most expensive single item in the operating budgets of many water utilities.[4]

The same situation exists at many Army installations that have their own water supply/distribution systems. These energy costs are especially devastating in light of the continual budget cuts within the military services. Moreover, the Army has directed its elements to explore every feasible option for reducing energy consumption.[5]

It is apparent that a major effort to reduce the electrical consumption and associated cost at water utilities should focus on efficient operation of raw and finished water pumps. This reduction could be accomplished by optimal real-time control of water supply pump stations. Optimal control strategies for water distribution system operation could be formulated using powerful optimization techniques (Chapter 2 presents a literature survey on this technology.) To be effective, such a method would allow operators to schedule pump operations such that the electrical consumption of the entire pump station is minimized while adequate system storage for fire protection and sufficient system pressures are maintained.

Optimal control of water supply systems may be welcome to plant operators in both the Government and private sector. According to a 1984 survey by an American Water Works Association (AWWA) committee, many utilities would like to see a progression towards computer-assisted or even computer-controlled operation of water systems.[6] The desire for such a movement is due in part to (1) the

---

[1] D. A. Dreyfus, "Persistent National Energy Issue," *ASCE Journal of Professional Issues in Engineering*, Vol 114, No. 1 (1988), pp 1-8.

[2] D. Brailey and A. Jacobs, "Energy Management in the Water Works Industry," *Journal of the New England Water Works Association*, Vol 94, No. 3 (1980), pp 216-239.

[3] J. L. Patton and M. B. Horsley, "Curbing the Distribution Energy Appetite," *Journal of the American Water Works Association (JAWWA)*, Vol 77, No. 6 (1980), pp 314-320; H. F. Rehis and M. K. Griffin, "Energy Costs Reduction Through Operational Practices," *Proceedings of the AWWA Annual Convention*, Dallas, TX (American Water Works Association [AWWA], 1984.

[4] W. H. Clingenpeel, "Optimizing Pump Operating Costs," *JAWWA*, Vol 75, No. 10 (1983), pp 502-509; T. Clark, "Reducing Power Costs for Pumping Water," *Opflow*, Vol 13, No. 10 (1987).

[5] Army Regulation (AR) 11-27, *The Army Energy Plan* (July 1989).

[6] AWWA Computer Assisted Design of Water Systems Committee, "Network Analysis Survey 1984," *Proceedings of the AWWA Annual Conference*, Dallas, TX (1984).

increased complexity of distribution systems, (2) high operating costs, (3) computer hardware which is becoming cheaper, faster, and more readily available, (4) managers and operators who are less threatened by computers than in past years, and (5) more sophisticated simulation models, optimization algorithms, and control software.[7]

Objective

The objective of this work was to develop and test a computerized method that will enable Army water utility managers and operators to select an optimal control strategy for water supply pump stations.

Approach

To develop the optimal control system, the literature was surveyed for existing technologies that might be useful (Chapter 2). Two that appeared promising (a nonlinear optimization algorithm and a simulation routine) were combined to form the model (Appendix A). Two subroutines were written next: one for data entry and another to provide computational features (Appendices B and C, respectively). A second subroutine for data entry was also written. The resulting program is called Optimal Control of Pumping Stations (OCOPS). It is written in FORTRAN 77 and can be used on any IBM-compatible personal computer (PC).

To determine this strategy's effectiveness, OCOPS was tested at the Fort Hood, TX water distribution system. The results were analyzed in terms of OCOPS' ability to reduce energy costs by suggesting schedules for pumping at nonpeak periods.

Scope

The program developed can be applied to any distribution system and is not limited to any number of pumps, pump stations, or tanks within the system. It provides an optimal control strategy based on electricity unit charges, but can be modified to account for electrical demand charges as well by including additional constraints.

Mode of Technology Transfer

It is recommended that the information in this report be transferred using both an information exchange and implementation package. The information exchange would consist of:

1. Briefings on the program and reports in field experience to be reported at the PROSPECT course entitled **Water Supply Design and Rehabilitation**, which is widely attended by DEH, MACOM, and Corps District personnel, and

2. The DEH Digest.

[7] U. Shamir, "Computer Applications for Real-Time Operation of Water Distribution Systems," *Proceedings of the ASCE Specialty Conference on Computer Applications in Water Resources*, H. C. Tomo (Ed.) (American Society of Civil Engineers [ASCE], 1985).

The implementation package would include:

1. A fact sheet for distribution at the DEH conference and a recommendation that interested DEH utility personnel attend the PROSPECT course entitled **Water Supply Design and Rehabilitation**.

2. A computer program user guide including specifications for gathering data for program inplementation.

It is also recommended that this computer program be further demonstrated through either the Facility Engineering Application Program (FEAP), or via a Technology Transfer Test Bed (T^3B).

The proponent agency for this computer program is the U.S. Army Engineering and Housing Support Center (USAEHSC). The POC at USAEHSC is the Sanitary Branch, at (703) 355-7963. Technical details on the use of this computer program can also be obtained from the U.S. Army Engineer Waterways Experiment Station (USAWES), phone (601) 634-2879 or toll free at (800) 522-6937, extension 2879.

2 SYSTEM DEVELOPMENT

Literature Review

Much of the previous research on reducing electrical consumption or increasing the efficiency of pump operation in water distribution systems has relied on heuristic approaches. Such general measures to improve energy efficiency include the use of variable-speed pumps, pumping during periods of low electrical costs, use of diesel-powered pumps during peak demand periods, improving driver power factors by installing capacitors, reducing the number of pump starts and shutdowns, and reducing pump heads by cleaning and lining mains and opening partially closed valves.[8]

Other researchers have employed mathematical optimization techniques in an effort to develop optimal control strategies. Most of this effort has focused on using dynamic programming since storage tank operation is well suited to state-stage discretization.[9]

The efficiency of dynamic programming deteriorates rapidly with an increase in the number of storage tanks in the distribution system. Some researchers have suggested decomposing the system into subsystems with one or two storage tanks and solving the optimal control program using dynamic programming.[10]

Other investigators have used nonlinear optimization techniques to solve the optimal control problem. These techniques are applied to simplified representations of the water distribution system's dynamics and are useful with highly dimensional systems.[11] Cohen provides a very good summary of the methods and algorithms developed for optimal control of water supply pump stations.[12]

Very little work has been done to integrate optimization algorithms with water distribution analysis

---

[8] G. A. Aldworth, "Energy-Saving Pump Selection," *JAWWA*, Vol 75, No. 10 (1983), pp 496-501; D. Brailey and A. Jacobs; J. Chao, "Can Off Peak Pumping Cut Utility Power Costs," *JAWWA*, Vol 71, No. 5 (1979), pp 259-263; T. Clark; W. H. Clingenpeel; G. W. Lackowitz and P. J. Pretretti, "Improving Energy Efficiency Through Computer Modeling," *JAWWA*, Vol 75, No. 10 (1983), pp 510-515; J. L. Patton and M. B. Horsley; H. F. Reheis and M. K. Griffin; B. S. Aptowicz et al., "Using Elevated Storage and Off-Peak Pumping to Control Energy Costs," *JAWWA*, Vol 79, No. 11 (1987).

[9] B. Coulbeck and C. H. Orr, "Optimized Pumping in Water Supply Systems," *Proceedings of IFAC, IX Triennial World Congress*, Budapest, Hungary, Vol 6 (1984); L. W. Ormsbee, et al., *Techniques for Improving Energy Efficiency at Water Supply Pumping Stations*, Technical Report EL-87-16 (U.S. Army Waterways Experiment Station, 1987); M. H. Sabet and O. J. Helweg, "Cost Effective Operation of Urban Water Supply System Using Dynamic Programming," *Water Resources Bulletin*, Vol 21, No. 1 (1985), pp 75-81; U. Shamir; J. L. Solanas and M. Verges, "Extension of the Dynamic Programming Successive Approximation and Its Application to Automatic Operational Control of Water Distribution Systems," *IFAC/IFORS Symposium*, Varna, Bulgaria (1974); M. J. H. Sterling and B. Coulbeck, "A Dynamic Programming Solution of Optimization of Pumping Costs," *Proceedings of the Institute of Civil Engineers*, Vol 59 (1975), pp 813-818; A. J. Tarquin and J. Dowdy, "Optimal Pump Operation in Water Distribution," *ASCE Journal of Hydraulic Engineering*, Vol 115, No. 2 (1989), pp 158-168.

[10] B. Coulbeck and C. H. Orr, "Optimized Pumping in Water Supply Systems," *Proceedings of the 3rd IFAC Symposium on Control of Distributed Parameter Systems*, Toulouse, France (1982).

[11] Cembrano, et al., "Optimization of a Multi-Reservoir Water Network Using a Conjugate Gradient Technique. A Case Study," *8th International Conference on Analysis and Optimization of Systems* (Institut National de Recherche en Informatique et en Automatique, Antibes, France, 1988); B. Coulbeck, "Optimal Operations in Non-Linear Water Networks," *Optimal Control Applications and Methods*, Vol 1 (1980), pp 131-141; B. Coulbeck and M. J. H. Sterling, "Optimized Control of Water Distribution Systems," *Proceedings of IEE*, Vol 125, No. 9 (Institute of Electrical Engineers [IEE], 1978), pp 1039-1044; F. Fallside and P. F. Perry, "Hierarchical Optimization of a Water-Supply Network," *Proceedings of IEE*, Vol 122, No. 2 (1975), pp 202-208; M. J. H. Sterling and B. Coulbeck, "Optimization of Water Pumping Costs by Hierarchical Methods," *Proceedings of the Institute of Civil Engineers*, Vol 59 (1975), pp 789-797.

[12] G. Cohen, "Optimal Control of Water Supply Networks, *Optimization and Control of Dynamic Operational Research Models*, S. G. Tzafetas (Ed.) (North Holland Publishing Co., 1982), pp 251-276.

packages. Although a few models have been developed using this procedure,[13] they consider only optimal design and not optimal operation.

Developmental Approach

The optimal control method developed in this study is conceptually very simple. A nonlinear optimization algorithm, GRG2,[14] was coupled with a water distribution simulation routine, WADISO.[15] The decision variable, or quantity that can be changed to achieve least-cost pumping, is the fraction of time a pump is operated during a given time interval. The nonlinear optimizer and the simulation routine were linked together by a subroutine called GCOMP which computes the cost of operation and updates all tank water levels.

GRG2 systematically adjusts the pump operating period during a given time interval and provides this information to the GCOMP routine. Based on this information, GCOMP finds the boundary conditions and passes this information to WADISO, which solves the equations describing the pressure and flow distribution in a water system. WADISO returns values for pump head and discharge for each operating pump, flow in lines connecting tanks to the system, and pressure at critical nodes to the GCOMP subroutine. GCOMP computes the electrical consumption and resulting operating cost for the specified time period; it also updates tank levels. This information is returned to GRG2 where it is used to generate an improved operating strategy, and the process is repeated until an optimal solution is reached. Figure 1 illustrates this process.

To test OCOPS, an experimental run was to be conducted at Fort Hood, TX. Chapters 3 and 4 describe the Fort Hood water distribution system and provide details on the computer model and the test application. Appendix A contains a detailed description of the solution methodology; GRG2, WADISO, and OCOPS also are explained further in this section.

---

[13] L. Ormsbee and D. Contractor, "Optimization of Hydraulic Networks," *International Symposium on Urban Hydrology, Hydraulics, and Sediment Control*, Lexington, KY (1981); K. E. Lansey, *Optimal Design of Large Scale Water Distribution Systems Under Multiple Loading Conditions*, Ph.D. Dissertation (University of Texas at Austin, 1987).

[14] L. S. Lasdon and A. D. Waren, *GRG2 User's Guide* (University of Texas, 1986).

[15] J. Gessler and T. M. Walski, *Water Distribution System Optimization*, Technical Report EL-85-11 (U.S. Army Engineer Waterways Experiment Station, 1985).

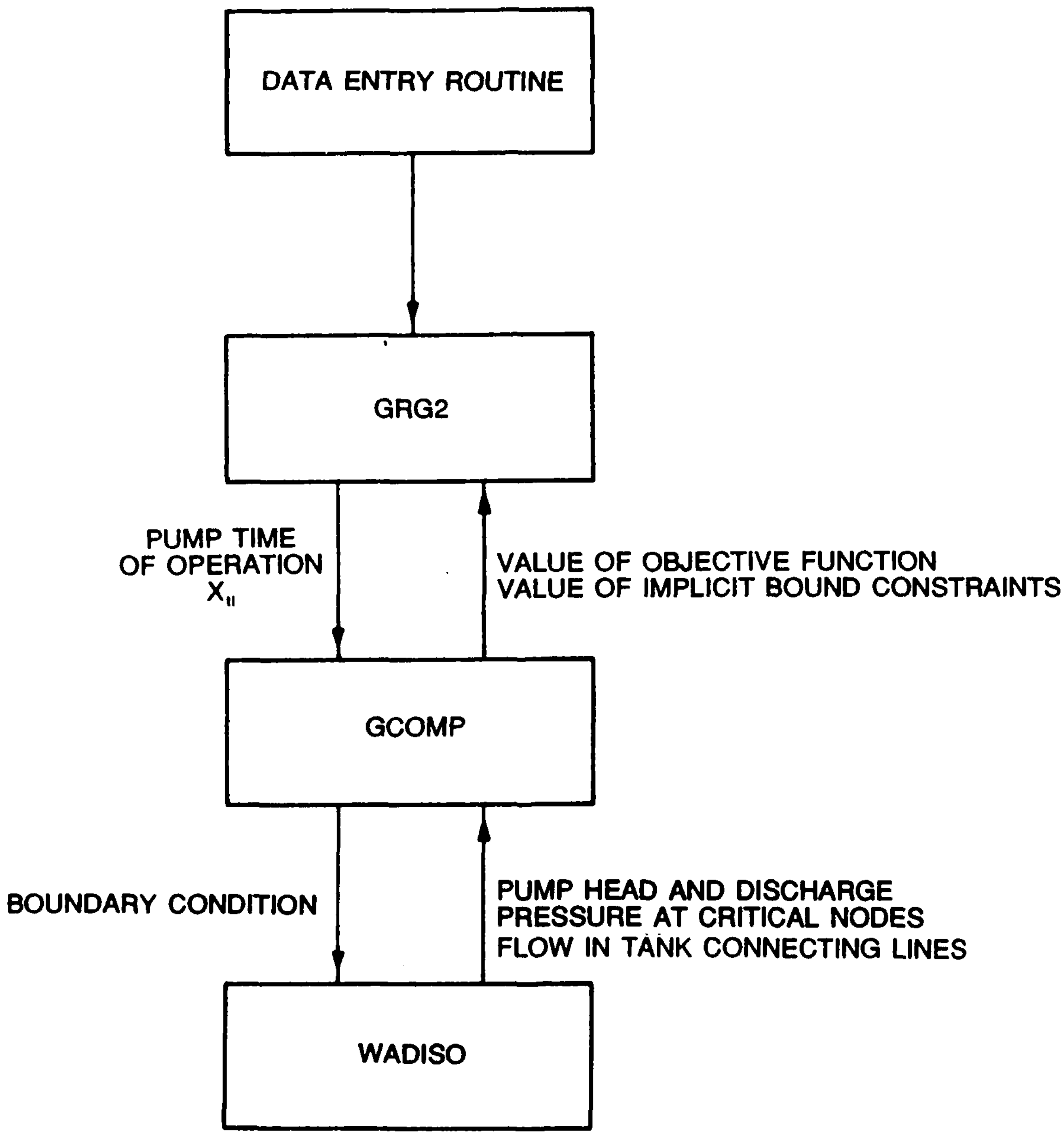

Figure 1. Optimal control process.

3 DESCRIPTION OF THE FORT HOOD WATER DISTRIBUTION SYSTEM

Fort Hood is located in east central Texas approximately 60 miles north of Austin. The installation serves a population of more than 64,000 and is one of the largest military installations in the United States. The post is divided into three general areas: North Fort Hood, training areas, and the Main Post. The Main Post contains the major fraction of total population, building space, and installation utilities. Therefore, only the distribution system serving the Main Post was included in the analysis.

Water Source

Water is purchased from the Bell County Water Control and Improvement District (WCID) and is delivered to Fort Hood through a storage and interconnection facility adjacent to the installation's main pump station. Two 5 million gallon (MG)* ground storage tanks at the facility supply water to Fort Hood. Water flows from the storage tanks through metered interconnections to the main pump station and also to three ground storage tanks located next to the pump station.

Main Pump Station

Water is delivered to the Main Post through five vertical turbine pumps. Table 1 lists the rated characteristics of each pump. Pump operation is controlled automatically by the water level in tank 5 in the western part of the main cantonment area. See Table 2 for level settings of each pump.

Table 1

Fort Hood Main Pump Station Pump Characteristics

Pump No.	Serial No.	Type	Rated Horsepower (hp)	Rated Head (ft)	Rated Discharge (gpm)
1	Verti-Line D06876	Vertical Turbine	100	262	1,150
2	Verti-Line D06760	Vertical Turbine	200	262	2,350
3	Fairbanks-Morse P2E3238	Vertical Turbine	300	262	3,500
4	771H0437 270	Vertical Turbine	500	262	6,000
5	Verti-Line 101449	Vertical Turbine	500	262	6,000

*A metric conversion table appears on p 43.

The values shown in Table 2 represent the summer pumping policy. This pumping sequence results in the 200-hp pump starting first, followed by the 300-hp, 100-hp, new 500-hp (pump 5), and old 500-hp (pump 4), respectively. The winter pumping sequence is the same as the summer's; however, the tank elevations are slightly different. Basically, the pumps are operated so as to keep all storage tanks as full as possible. Figure 2 is a detail of the main pump station and WCID interconnection.

Distribution System

The Main Post distribution system contains approximately 330 miles of pipeline ranging in size from 6 to 30 in., with smaller individual service lines. Most of the lines in the Main Post area are unlined cast iron, whereas asbestos cement and ductile iron pipe carry water to family housing areas. Six elevated steel tanks provide a total system storage of 5 MG, and are located at various points within the distribution system. Table 3 describes each tank. Figure 3 is a schematic of the Fort Hood distribution system analyzed in this study.

Water Demands

Water consumption at Fort Hood varies seasonally, with peak usage occurring during the summer. Table 4 shows the average daily water demand per month between March 1987 and February 1988. Increased water consumption during the summer months is due in part to irrigation of athletic fields, a golf course, and lawns in the family housing areas. Increased military training also accounts for added summer usage.

Table 2

Fort Hood Main Pump Station Tank Level Settings*

	Level To Turn Pump On		Level To Turn Pump Off	
Pump No.	(ft)	(Elev.)**	(ft)	(Elev.)**
1	31	1078.2	35	1082.2
2	33	1080.2	40	1087.2
3	32	1079.2	39	1086.2
4	22	1069.2	25	1072.2
5	25	1072.2	33	1080.2

*Levels shown represent pumping policy for summer season.

**Elevation to the top of the water in the tank in ft mean sea level (msl).

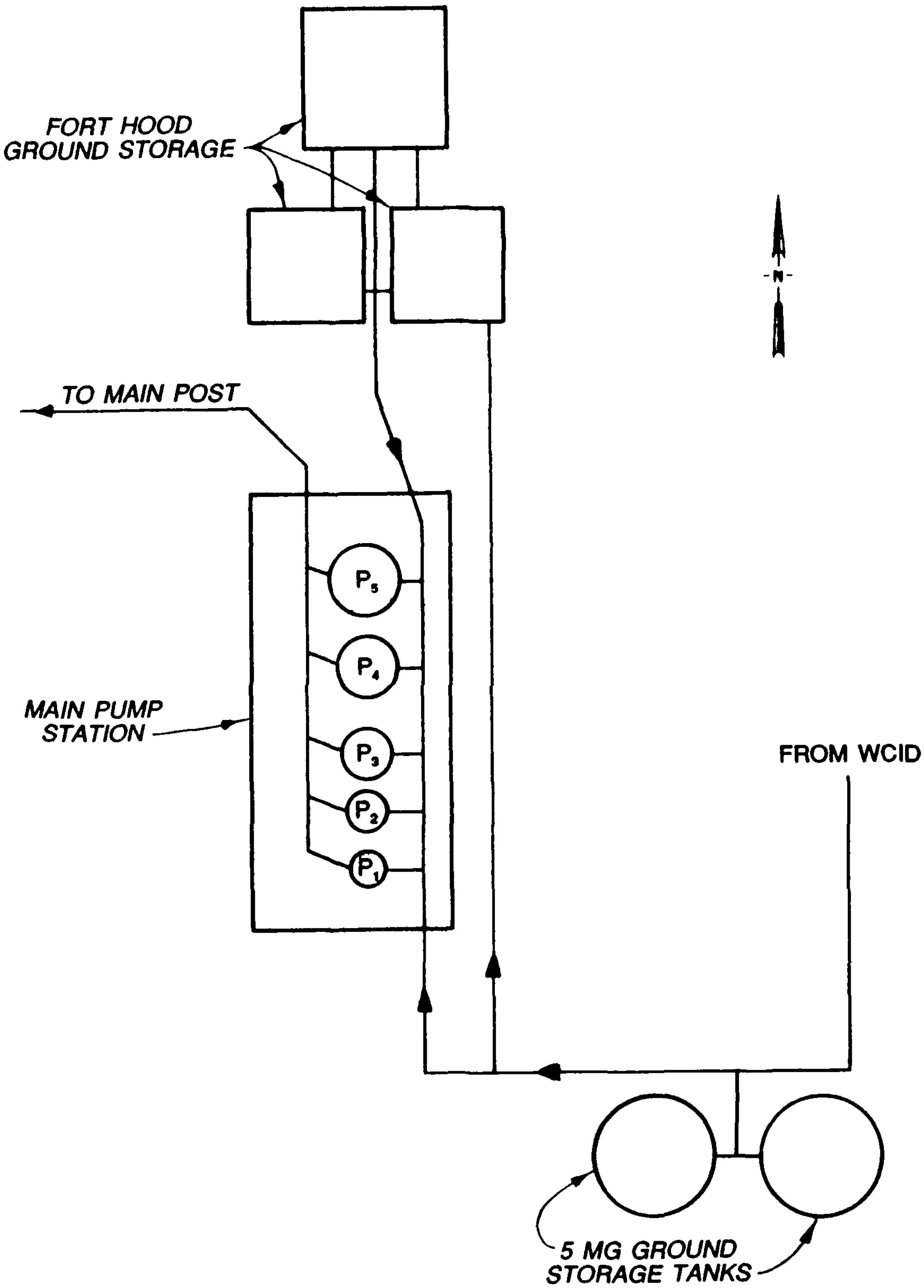

Figure 2. Fort Hood ground storage and pumping facility configuration.

Table 3

Fort Hood Storage Tank Description

Tank No.	Node No.*	Location	Capacity (1000 gal)	Max Elev.**	Min Elev.**
1	25	Pershing Park	500	1086.34	1049.00
2	32	East Post	1000	1086.34	1051.59
3	39	Mid Post	500	1086.34	1047.17
4	45	Railhead	1500	1086.34	1051.34
5	50	West Post	500	1086.34	1047.17
6	57	Comanche Village	1000	1086.34	1051.34
		Total Storage:	5000		

*See Figure 3.
**In ft msl.

Water consumption at Fort Hood also varies temporally with increased water usage during the midday and evening hours. To identify the diurnal loading pattern, tank water level variations were examined under known operating conditions (i.e., which pumps were running). Only those days for which the optimal control was applied (30 July 1988 and 1 August 1988) were analyzed for temporal water variation. Table 5 lists the percentage of average daily demand distributed over 4-hr periods for each day analyzed.

Computer Model

The optimal control methodology combines a nonlinear optimization algorithm with a simulation routine in an attempt to achieve least-cost pumping. The simulation routine, WADISO, solves the equations of continuity and energy associated with water distribution networks. A mathematical model of the Fort Hood water distribution system was constructed so that pump heads and discharges, tank water levels, and pressure at system nodes could be determined.

System Schematic

Before a computer program could be developed, a schematic of the network had to be devised. The resulting schematic illustrates the network through the use of circles and line segments. Each line segment corresponds to an individual pipe in the distribution system. Each circle represents a system node which is either the location where two or more pipes intersect or where demands are withdrawn from the system.

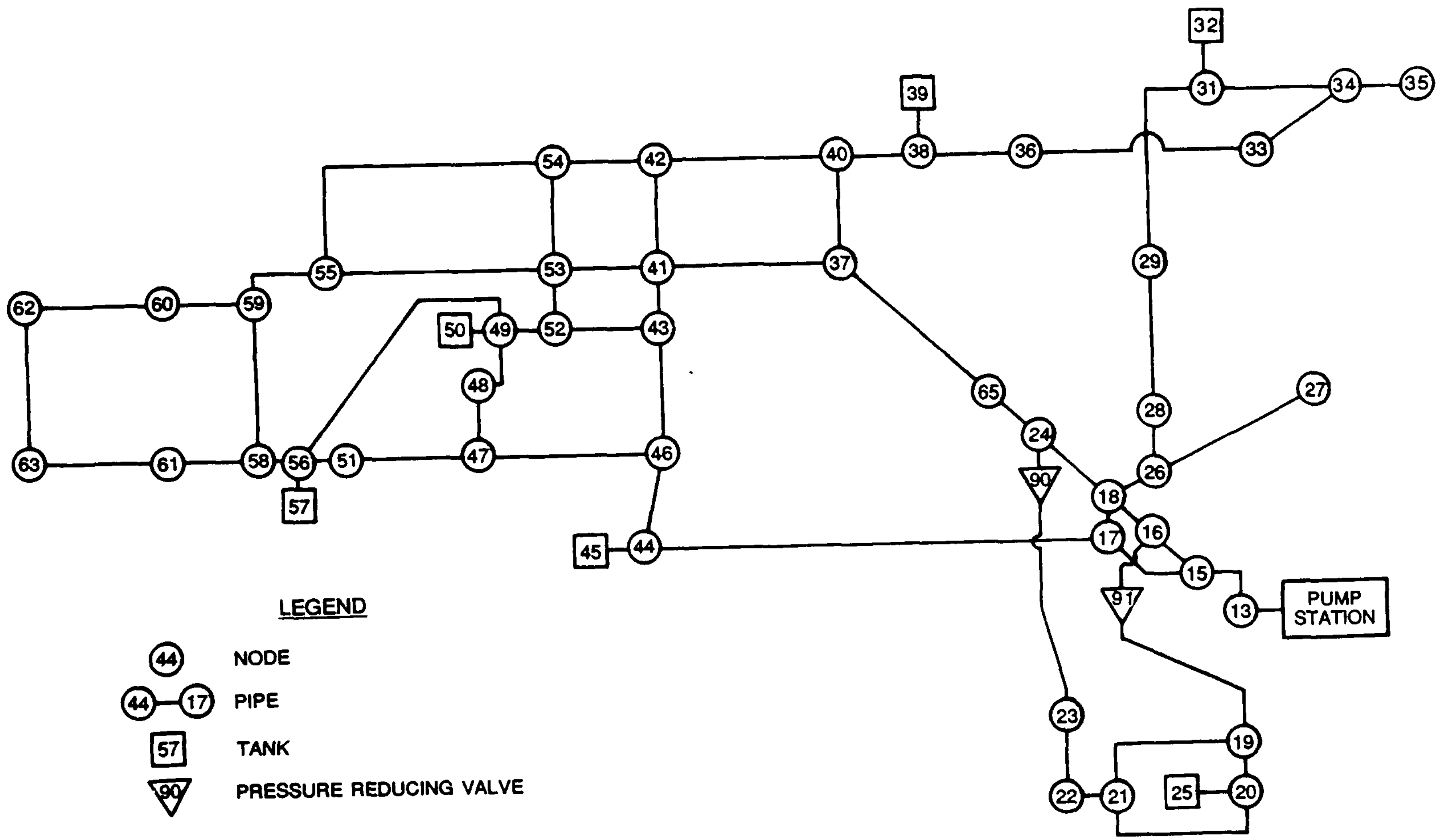

Figure 3. Fort Hood water distribution system configuration.

Table 4

Average Daily Demand per Month

Month/Year		Avg. Daily Demand (gpm)
March	1987	25,937
April	1987	7,647
May	1987	7,806
June	1987	8,335
July	1987	10,293
August	1987	12,903
September	1987	7,895
October	1987	7,554
November	1987	6,188
December	1987	5,613
January	1988	6,821
February	1988	5,854

Table 5

Temporal Distribution of Average Daily Demand

	30 July 1988		1 Aug 1988	
Time (hrs)	Water Consumed %	Global Demand Factor	Water Consumed (%)	Global Demand Factor
0000-0400	13.25	1.4753	12.71	1.9358
0400-0800	16.76	1.8653	16.85	2.5661
0800-1200	20.83	2.3189	20.39	3.1062
1200-1600	17.39	1.9364	18.11	2.7587
1600-2000	17.72	1.9721	18.93	2.8829
2000-2400	14.05	1.5642	13.01	1.9806

In computer modeling, it is usually not necessary to include every pipe in the distribution system. Instead, it is possible to analyze a skeletal system that includes only the major mains. Results of a skeletal model are still accurate because the mains not included in the skeletal model do not carry a great deal of flow.[16] Figure 3 is a schematic of the computer model developed for the Fort Hood system.

[16]T. M. Walski, *Analysis of Water Distribution Systems* (Van Nostrand Reinhold, 1984).

Network Data

Once a network schematic of the system was developed, a data base had to be established. The data base required information on each pipe and node included in the schematic, and on all pumps, tanks, and flow control valves. In addition, the demands associated with the various nodes had to be determined.

Pipe Data

Maps showing the locations of all mains in the system were obtained from the Fort Hood Directorate of Engineering and Housing (DEH). Pipes to be included in the model were selected based on size and importance. Lengths between nodes were scaled off the maps and recorded. Roughness values were determined by field tests and calibration analysis. All pipe data used in the analysis are listed in Appendix D.

Node Data

Nodes were placed at locations where two or more pipes join together. Node data requirements included elevation and water use. Elevations for nodes in the Fort Hood system were taken from contour maps, with contours spaced at 5-ft intervals. Daily water consumption was estimated from historical records provided by the Fort Hood DEH. Hourly water consumption percentages were obtained from an analysis of the operating conditions at the installation.

In actual operation, water is withdrawn from a water main at various service connections located along the entire length of the pipe. However, when modeling a distribution system, these demands are aggregated at individual nodes in the immediate vicinity of water withdrawal. For example, the combined water demand for an entire family housing area may be placed at a single node.

In modeling the Fort Hood distribution system, the average daily demand was distributed among several nodes. The selection of demand nodes and the amount of water distributed to each node were based on an analysis of major users at the Main Post. These users included family housing, barracks, washracks, the east post airfield, and Darnall Hospital. A complete set of node data is provided in Appendix D.

Tank Data

Six storage tanks were included in the computer model. All tanks operate in the same pressure zone and have the same overflow elevation (Table 3).

PRV Data

At present, the Fort Hood system contains two pressure-reducing valves (PRVs) isolating the Pershing Park and Venable Village areas (nodes 19 through 23 in Figure 3) from the rest of the system. The PRVs are necessary to reduce the pressures in these areas since they lie at low elevations with respect to the rest of the system and are close to the main pump station. The hydraulic grade of both PRVs was set at 1101 ft.

Pump Field Tests

As part of the optimal control study, the pumps at the main pump station were field-tested to determine their actual operating characteristics. The testing was conducted on 17 and 18 March 1988. Each pump was tested individually and, in some cases, parallel combinations of pumps were tested to obtain data over a wide variety of operating conditions.

The main pump station contains five vertical turbine pumps. Each pump is rated at 262 ft of head (Table 1), but the discharges vary. All pumps pull water from a common suction line and discharge water into a single discharge header.

Testing Procedure

For each test, the pump was started against a closed valve in the discharge line to obtain a shutoff head. After the pump reached full speed, a pressure reading was taken from a gauge located in the discharge line between the pump and the valve. A pressure gauge installed at the base of the old 500-hp pump provided a reading for suction head. The difference in pressure between the two gauge displays was the head delivered by the pump. The power consumption of the pump was also recorded. Once shutoff head and corresponding power were obtained, the valve was opened slightly and pressure, discharge, and power consumption values were recorded. This process was repeated, with the valve opening increased each time until the valve was completely open.

Pump discharge was measured using a pitot tube inserted in the pump discharge header. The pitot tube provided a pressure differential that was converted into pipeline velocity and ultimately into pipe flow rate. Power readings in the form of current draw and voltage drop across the pump were taken and converted into kilowatt consumption.

Pump Data

For this analysis, only the pumps located at the main pump station were included. Although none of the booster pumps located at Fort Hood were considered in the study, the proposed methodology can easily handle such situations. To include booster stations, the pump's characteristics must be known and they must use electrical power.

A careful review of the data indicated that nearly all of the pumps have characteristics close to the manufacturers' original specifications. As a result, the pump data provided by the manufacturers were used in the analysis. The old 500-hp pump had pump heads and discharges different from those shown by the original curve. Consequently, this pump was omitted from the analysis.

System Calibration

When using a computer simulation routine to analyze a water distribution system, it is extremely important that the mathematical model of the system be an accurate representation of actual field conditions. If this is not the case, the results provided by the simulation routine will be of limited value. Therefore, the model must be calibrated. Calibration is done by adjusting both water usage and pipe roughness until heads and flows computed by the simulation routine match those observed in field measurements.

One common method of obtaining calibration data is to conduct fire hydrant flow tests at various locations throughout the system. These tests provide hydraulic information (pressures and flows) during both low flow and high flow periods. This information can be used with the network's computer model to calibrate the system.

Calibration data for the Fort Hood Main Post distribution system were gathered from fire hydrant flow tests conducted on 15 and 16 March 1988. The location of each fire hydrant flow test is shown in Figure 4; Table 6 lists results for each test. For each test, at least two and, in most cases, three hydrants were flowed to induce as much stress into the system as possible.

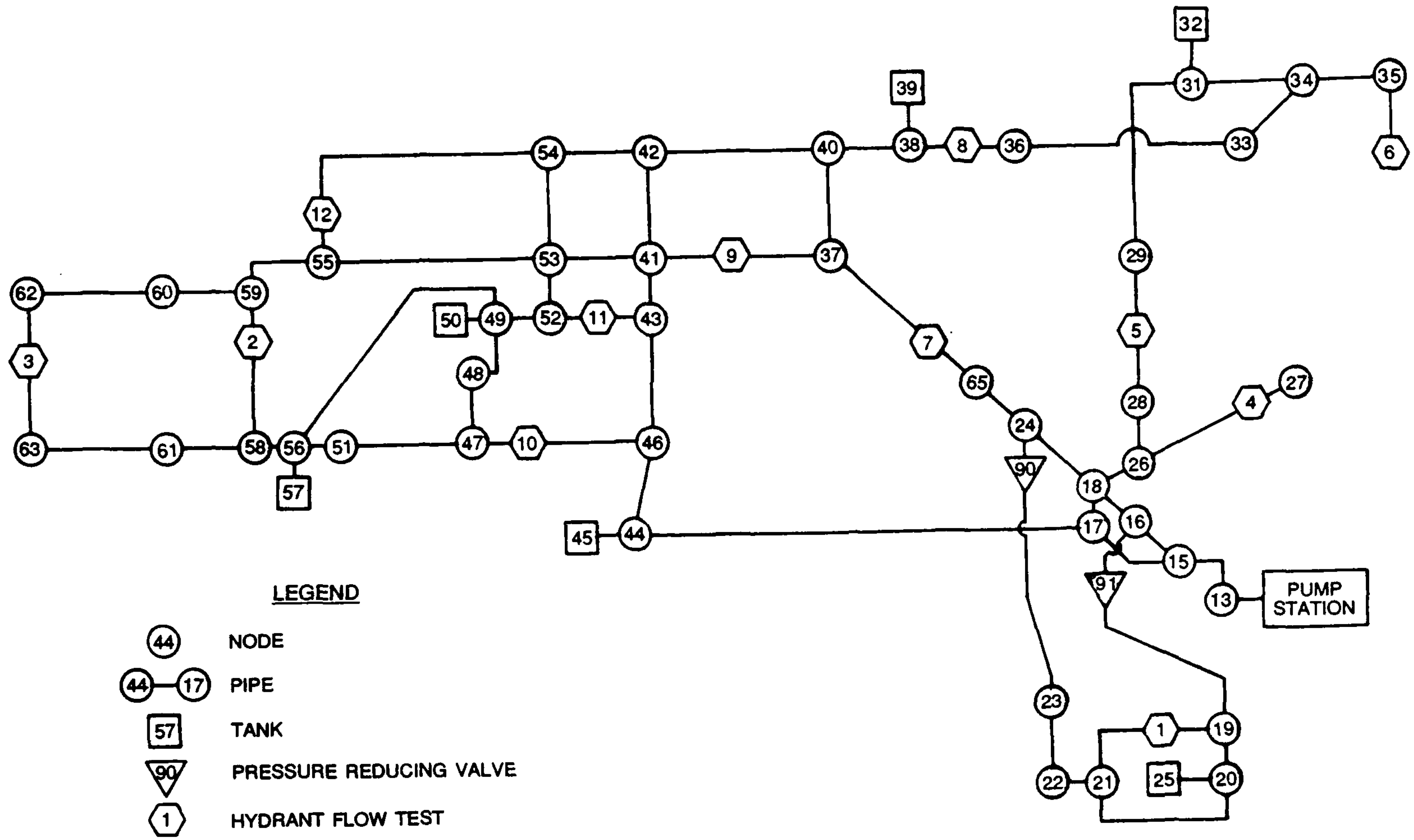

Figure 4. Locations of fire hydrant flow tests conducted at Fort Hood.

Table 6

Hydrant Flow Test Results for Fort Hood Main Post

Test No.	Location	Residual Node	Elev. (ft)	Number of Hydrants Flowed	Pressure (psi)	Combined Hydrant Discharge (gpm)
1	Pershing Park	70	878	0	89.5	0
				1	61.5	1288
				2	40	1922
				3	28	2386
2	Comanche Village	80	888	0	82	0
				1	77	1336
				2	70	2357
				3	62	3320
3	Duncan School	81	923	0	68	0
				1	58	1033
				2	38	1708
4	Anderson Golf Course	27	905	0	83	0
				1	54	903
				2	30	1254
5	Meadows School	72	906	0	83	0
				1	77	1277
				2	71	2509
				3	70	3717
6	East Fort Hood Airfield	35	891	0	84	0
				1	75	1351
				2	72	2120
				3	68	3283
7	McNair Village	75	912	0	84	0
				1	74	1220
				2	71	2451
				3	69	3384
8	Central Ave. between 37th and 42nd St.	74	924	0	69	0
				1	63	1185
				2	57	2112
				2	57	2112
9	Battalion Ave between 52nd and 62nd St.	76	937	0	62	0
				1	62	1060
				2	61	2184
				3	58	3318
10	Warehouse Rd. and 72nd St.	79	965	0	49	0
				1	46	1073
				2	41	1936
				3	37	2701
11	Support Ave. and 72nd St.	78	930	0	66	0
				1	65	1099
				2	64	2350
				3	63	3490
12	Battalion Ave. and 78th St.	55	898	0	80	0
				1	75	1320
				2	67	2542
				3	60	3495

The WADISO model of the water distribution system was calibrated using a nonlinear optimization technique.[17] The underlying theory of this technique is very similar to the optimal control methodology used to develop the system. The optimization code, GRG2 was coupled with a hydraulic simulation routine, WADISO, to minimize the difference between observed and predicted system heads and observed and predicted tank water levels. The decision variables in the model are pipe roughness, nodal water demand, and global demand factor.

Pipe roughness and nodal water demand are systematically adjusted within defined limits until the difference between observed and predicted heads is minimized. The global demand factor is a constant by which each nodal demand is multiplied to account for the hourly variation in water demand. This variable is also adjusted between defined limits until the difference between observed and predicted tank water levels is minimized.

The calibration technique allows the adjusted parameters to be constrained between upper and lower values which are selected based on a knowledge of the system. Table 7 compares the observed and predicted system heads after calibration for selected fire flow tests. Calibration results for the Fort Hood system are presented in Appendix D as Hazen-Williams C-Factor and Node Demand. Calibrated global demand factors are shown in Table 5.

Note that only one tank in the Fort Hood distribution system, tank 5, is monitored for variation in water level. Thus, information on the water levels for the remaining tanks was not available. For the global demand factor calibration, tanks 1 through 4 were assumed to be full at the beginning of the analysis, the level in tank 5 was known, and the level in tank 6 was assumed to be 5 ft less than the level in tank 5. These assumed starting elevations were chosen based on the results of the steadystate simulation of the Fort Hood system and on conversations with installation personnel.

The actual tank levels over the course of the day for all tanks other than tank 5 were obtained from the global demand factor calibration. The starting elevations for all tanks other than tank 5 were assumed. The global demand factor for a given time period was found by minimizing the difference between the known and predicted levels for tank 5. The ending elevation of each tank was recorded and used as the beginning level for the next time step. This procedure was repeated for each time step analyzed.

---

[17] W. W. Sharp and D. W. Chase, "Verifying Water Distribution Computer Model Calibration Results," *AWWA Computers and Automation Specialty Conference*, Denver, CO (1989).

Table 7

Calibration Results for Fort Hood Main Post

	Location	Number of Hydrants Flowed	Observed Head (ft)	Predicted Head (ft)
1	Pershing Park	0	1085	1088
		2	970	970
2	Comanche Village	0	1077	1079
		1	1066	1057
		2	1050	1047
		3	1031	1030
3	Duncan School	0	1080	1079
		1	1057	1048
		2	1011	1007
4	Meadows School	0	1098	1101
		1	1084	1087
		3	1068	1068
5	East Fort Hood Airfield	0	1085	1097
		1	1064	1077
		2	1057	1065
		3	1048	1041
6	McNair Village	1	1083	1090
		2	1076	1087
7	Central Ave. between 37th and 42nd St.	0	1083	1092
		1	1070	1075
		2	1056	1055
8	Battalion Ave. between 52nd and 62nd St.	0	1080	1089
		1	1080	1084
		2	1079	1081
		3	1071	1080
9	Warehouse Rd. and 72nd St.	0	1078	1080
		1	1071	1074
		2	1060	1064
		3	1050	1051
10	Support Ave. and 72nd St.	0	1082	1079
		1	1080	1077
		2	1078	1077
		3	1076	1076
11	Battalion Ave. and 78th St.	0	1083	1078
		1	1071	1067

4 APPLICATION OF THE OPTIMAL CONTROL METHODOLOGY

The optimal control methodology was applied to the Fort Hood water distribution system to determine the effectiveness of the model and demonstrate its capabilities. The methodology was applied to 2 days for which data on actual operating conditions were available. The days were 30 July 1988 (weekend) and 1 August 1988 (weekday).

A review of actual tank levels and pump operating times indicated that Fort Hood system demands were cyclic with a period of 1 day. The demands were approximately the same during the week from Monday to Friday. System demands were less on the weekends; however, the weekend demands were also cyclic with a daily period. For this reason, 2 days, one during the week and the other on the weekend, were selected. A review of climatic data indicated that precipitation and temperature were normal for several days up to and including the days analyzed.

Electrical Rate Structure

Fort Hood purchases electricity from Texas Utilities Electric Company under a general service rate schedule at an average cost of $0.045/kWh. The contract between Fort Hood and the Texas Utilities Electric Company contains a time-of-day option; however, this option applies only to electrical demand charges and not to energy unit charges. For this study, the average cost of electricity is applied evenly over the course of the day.

Actual Operating Results

Pump operation at Fort Hood is controlled by the water level in tank 5 or the West Post Tank. Therefore, this is the only tank in which the water levels are monitored. Analysis of the calibration results indicated that tanks 1 and 4 stay completely full during normal operating conditions. This was verified from conversations with Fort Hood personnel. During periods of high demand or when the 100-hp pump is operating independently, tanks 1 and 4 will drain. Because of its location, water must travel through many feet of small diameter pipe between the pump station and tank 6. The resulting head loss causes tank 6 to drain under normal operating conditions.

Actual Cost of Operations

The Fort Hood pump station does not have electric meters installed on the pumps. As a result, the actual electrical consumption and hence the operating cost cannot be computed directly. The actual operating cost used in this study was computed based on the known pump operating times, the known tank trajectory for tank 5, and the calibrated tank trajectories for the remaining tanks. As a result, the cost is not a true cost of operation, but is a computed cost based on available information.

30 July 1988

The water level in tank 5 at midnight was 36.12 ft and dropped to a low level of 31 ft in the early morning and late afternoon hours. The water level at the end of the day was 32 ft. During the day, the 100-, 200-, and 300-hp pumps were operated. Table 8 lists the pump operating times and corresponding tank water levels for tank 5. Note that pump 3 cycles on and off approximately every hour during the early morning hours. The actual cost of operation for Saturday, 30 July 1988, was $380.36.

Table 8

Actual Pump Operating Times for 30 July 1988

Time (hr)	Beginning Elevation (ft)	Ending Elevation (ft)	Pumps Operating
0000-0030	36.12	32.00	2
0030-0145	32.00	39.00	2,3
0145-0230	39.00	32.00	2
0230-0345	32.00	39.00	2,3
0345-0445	39.00	32.00	2
0445-0900	32.00	31.00	2,3
0900-1500	31.00	35.00	1,2,3
1500-1830	35.00	31.00	2,3
1830-2120	21.00	35.00	1,2,3
2120-2300	35.00	39.00	2,3
2300-2400	39.00	32.00	2

1 August 1988

The beginning and ending water levels for tank 5 were 35.18 and 35.00 ft, respectively. The low water level for the day was 25 ft. The 100-, 200-, 300-, and new 500-hp pumps were operated at various times during the day. Table 9 lists the pump operating times and associated tank levels for tank 5 on Monday, 1 August 1988. As is the case on 30 July 1988, pumps (3 and 5 in this case) cycle on and off several times during the day. The actual cost of operation for this day was $560.75.

Optimal Operating Results

OCOPS was applied to the 2 test days to determine if the actual operating procedure at Fort Hood could be modified to achieve a cost of pumping less than the actual cost while still maintaining sufficient system pressure and adequate system storage. The time interval selected for analysis was 4 hr. Constraints were placed on the system's pressures and storage. These constraints were evaluated at the end of each time interval.

Implicit Bound Constraints

Implicit bound constraints (Appendix A) were placed on system pressure and storage. The pressure constraint was applied to nodes 24, 42, and 47. These nodes were selected based on their elevation and location within the distribution system. Analysis of the calibration results indicated that node 24 had the highest pressure (large diameter main close to the pump station) within the system. Nodes 42 and 47 had the lowest pressures (high elevation) in the system.

Nodes at the suction and discharge sides of the pumps were not included in the pressure constraint set since they were expected to have very low and high pressures, respectively. The maximum allowable pressure was set at 100 psi while the minimum acceptable pressure was 40 psi.

Table 9

Actual Pump Operating Times for 1 August 1988

Time (hr)	Beginning Elevation (ft)	Ending Elevation (ft)	Pumps Operating
0000-0309	35.18	39.00	2,3
0309-0340	39.00	32.00	2
0340-0600	32.00	31.00	2,3
0600-0700	31.00	25.00	1,2,3
0700-0830	25.00	33.00	1,2,3,5
0830-0930	33.00	25.00	1,2,3
0930-1115	25.00	33.00	1,2,3,5
1115-1200	33.00	25.00	1,2,3
1200-1345	25.00	32.00	1,2,3,5
1345-1510	33.00	25.00	1,2,3
1510-1615	25.00	33.00	1,2,3,5
1615-1725	33.00	25.00	1,2,3
1725-1900	25.00	33.00	1,2,3,5
1900-2000	33.00	25.00	1,2,3
2000-2100	25.00	33.00	1,2,3,5
2100-2400	33.00	35.00	1,2,3

Constraints were also placed on all tanks in order to keep a reserve of water in the system for fire-fighting needs and to have the tank levels at a desired ending elevation. The upper bound placed on the water level was the tank's top elevation. This point was selected so that the tank would not fill past its overflow elevation. The upper bound of the tank constraint equal to the top elevation of the tank was in force for each time interval.

The lower bound tank constraint was dependent on two items: (1) the amount of storage necessary for fire-fighting purposes and (2) the actual ending elevation of the tank. For all time intervals except the last one, the lower tank constraint depended on the storage necessary in the system. The available storage in the system during these time intervals was constrained to be no less than 1.25 MG. This amount is enough to combat a 5000-gpm fire for 4 hr. In most cases, the available storage was greater than 1.25 MG. Table 10 lists the upper and lower constraints placed on each tank.

For the last time interval, the lower bound tank constraint was set at the actual ending elevation of the tanks for the days analyzed. The optimization algorithm attempts to force tank levels to their lower bound during the final time interval since a lower tank level results in a lower pumping head, which results in lower operating costs. Thus, it is expected that the final tank levels from the optimal procedure would be equal or slightly above the final tank levels resulting from the actual procedure. This convention enables the actual and optimal operating policies to be compared equally.

A 4-hr time interval was selected to reduce the computer run times. The disadvantage to using larger time increments is that the method's flexibility is compromised. Pumps are actually operated over a

continuous timeframe as opposed to a discrete timeframe. The smaller the time interval, the closer the results approach continuous operation. Consequently, the smaller the time step, the greater the possibility for savings. Unfortunately, smaller time steps increase the time necessary to reach an optimal solution and could result in pumps cycling on and off frequently. Pump cycling is undesirable since it wastes energy and causes wear and tear on a pump.

30 July 1988

The optimization algorithm used in the model requires an initial guess to start the optimization process. The guess selected for this day was chosen to approximate the actual pump operating times. This choice resulted in pump 2 operating for the entire 24-hr period and pump 3 running for all but 4.5 hr. Pump 1 ran for about 8 hr while pump 5 did not operate at all. The cost of operation under the initial guess was $366.92. Figure 5 compares the actual operating times and the operating times corresponding to the initial guess. Note that, in the optimal control methodology, pumps are not allowed to begin operating in the middle of a time interval. Rather they must start operating at the beginning of the time interval and run continuously for $X_{t,i}$ hours.

Although the initial guess resulted in an operating cost less than the actual cost, the ending tank levels for each tank were less than the ending levels resulting from the calibration analysis. No pressure constraints were violated, however. The ending tank level constraint violation can be attributed to the initial guess only approximating actual pump operating times and to the global demand factor being a 4-hr average computed from calibrated global demand factors in effect throughout the day.

The optimization algorithm entered into a Phase 1 optimization to satisfy the violated tank constraints. The constraints were satisfied by increasing the pump run times for all pumps not already at a lower or upper bound. The cost of pumping after all constraints were satisfied was $392.02. Once all constraints were satisfied, the algorithm entered into a Phase 2 optimization to complete the optimization.

The cost of operation provided by the OCOPS upon completion of the Phase 2 optimization was $390.00. This cost is approximately $10 greater than the actual cost of operation. No constraints were violated under the final solution. Most of the ending elevations for the tanks were close to their lower

Table 10

Tank Water Level Constraints

Tank No.	Minimum Acceptable Water Level (ft)	Maximum Allowable Water Level (ft)
1	9.00	37.34
2	9.00	34.75
3	9.00	39.17
4	9.00	35.00
5	9.00	39.17
6	9.00	35.00

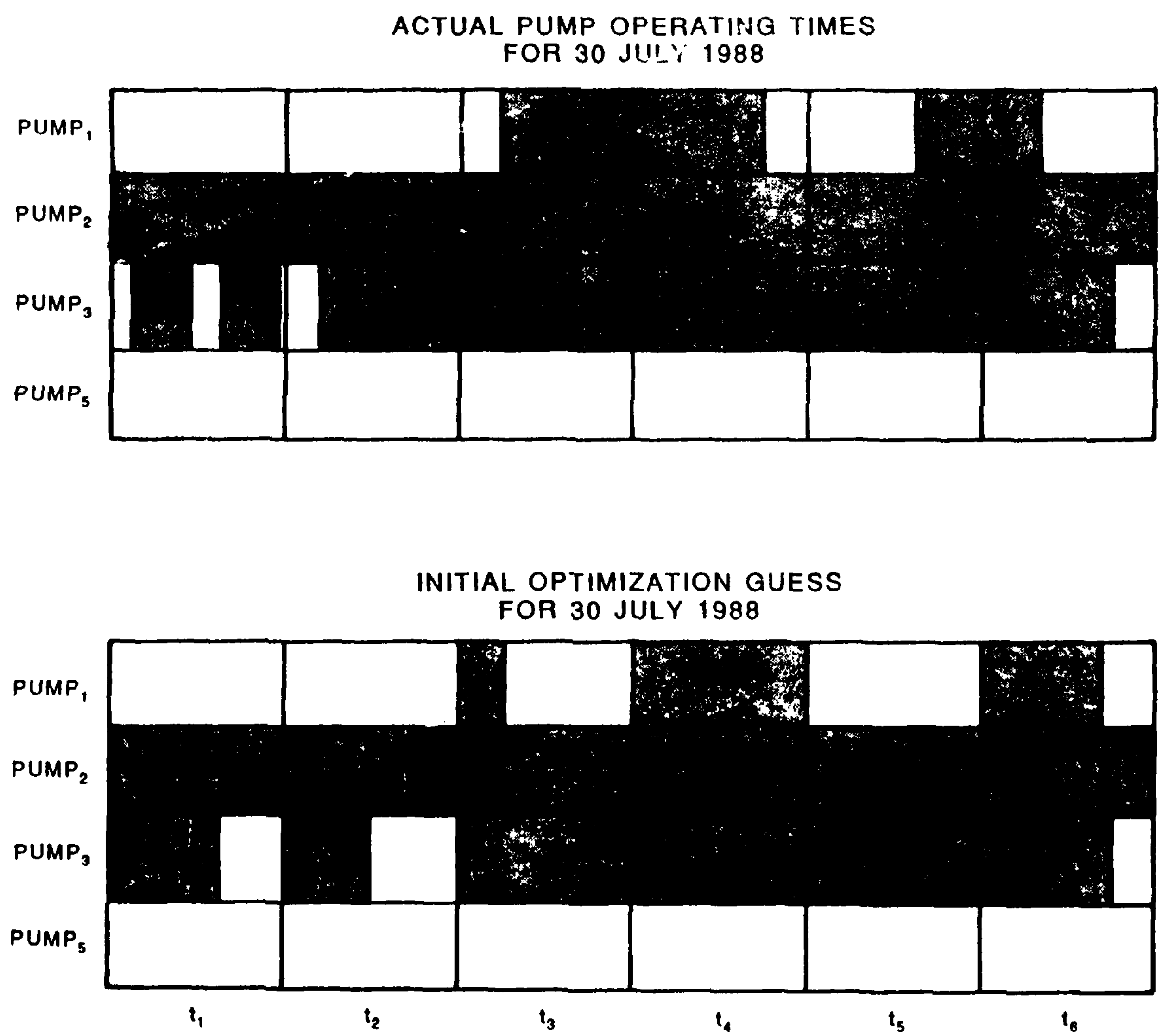

Figure 5. Actual and initial guess operating times for 30 July 1988.

bound. This result was expected because the nature of the problem causes tank levels to seek their lower bounds since pumping heads are a function of tank water level. The lower the tank level, the lower the pump head, and hence the lower the cost of operation. Figure 6 depicts the pump operating times for the final solution, and Figures 7 through 12 show the resulting tank trajectories for actual and optimal conditions.

1 August 1988

The initial guess provided for this day again was an approximation of the actual operating times for pumps. This guess resulted in pumps 2 and 3 operating for the entire day, pump 1 operating for 18 hr, and pump 5 running for about 8 hr. The cost of operation associated with the initial guess was $555.82. Figure 13 compares the actual operating times and the time associated with the initial guess for 1 August 1988.

As was the case with 30 July 1988, the initial guess resulted in the final water level in several tanks being below the desired ending elevation. As a result, the optimization algorithm entered into a Phase 1 optimization to satisfy the tank level constraints. This was accomplished by increasing the value of decision variables not already at a bound. No other pressure or tank level constraints were violated due to the initial guess.

The cost of pump operation provided by the optimal solution was $562.88 which was about $2 greater than the actual cost of pumping. Like the previous case, all constraints were satisfied and the ending tank water level for all tanks was either at or slightly above its lower bound. Figure 14 shows the operating times for each pump corresponding to the optimal solution. Figures 15 through 20 show the tank levels associated with actual and optimal operation.

Discussion

The benefit of applying mathematical optimization to pump operating strategies for water distribution systems has been demonstrated previously.[18] For example, an optimization approach using dynamic programming (DP) was applied to the Washington, DC system and resulted in projected annual savings of nearly $100,000.[19] DP works quite well for systems such as that in Washington which have only one or two storage tanks in a service area. However, most systems have more than one or two tanks in a service area and as a result, the use of DP becomes infeasible due to dimensionality. In this study, the nonlinear programming algorithm OCOPS was developed for use with highly dimensional systems such as the one at Fort Hood, TX.

All optimization models require accurate simulation of system response. In the case of pump optimization, the water distribution system response is described in the form of the pressure and flow variation within the network, the changes in tank level, and values for pump head and pump discharge. The modeled response must be very close to the actual system response for the optimization algorithm to operate correctly and the simulation routine to provide meaningful results. In other words, the simulation model must be well calibrated for the system being optimized.

As discussed earlier, the lack of information on tank level variation prevented complete calibration of the Fort Hood system. These data were available for one tank only at Fort Hood. Tank trajectories for the remaining tanks were not known. Therefore, the starting elevation for five of the six tanks had to be assumed and the system calibrated based on information for the one tank.

[18]W. H. Clingenpeel; A.J. Tarquin and J. Dowdy.
[19]L. Ormsbee.

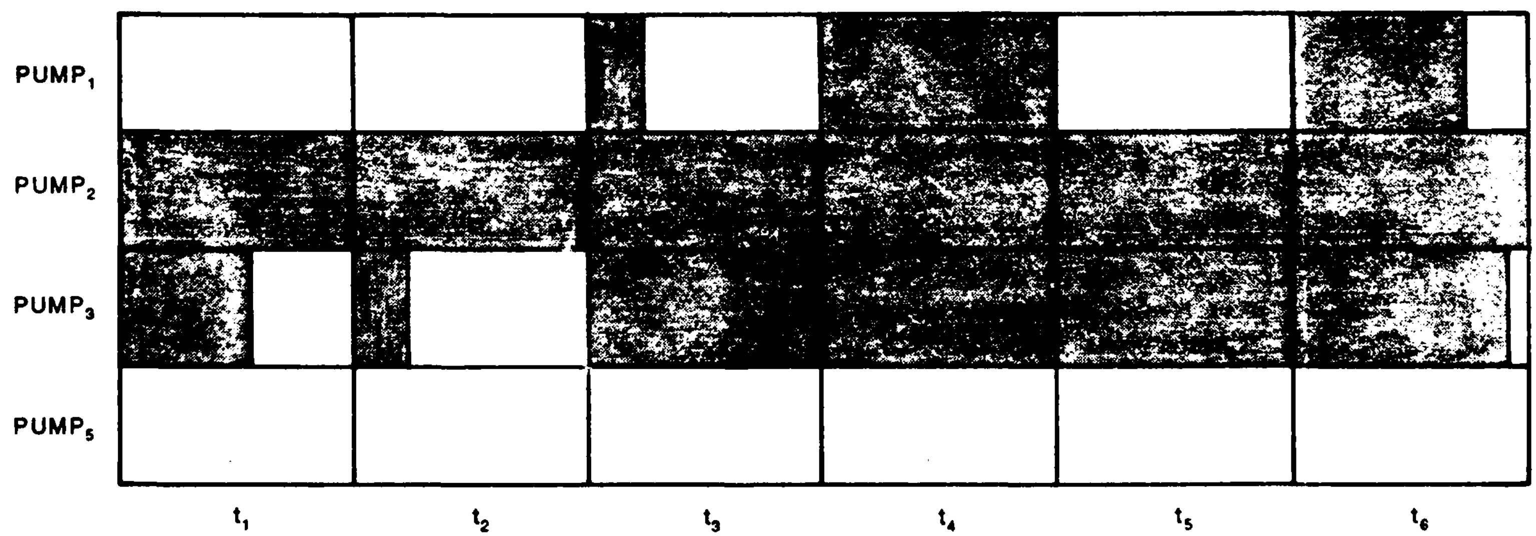

Figure 6. Optimal pump operating times for 30 July 1988.

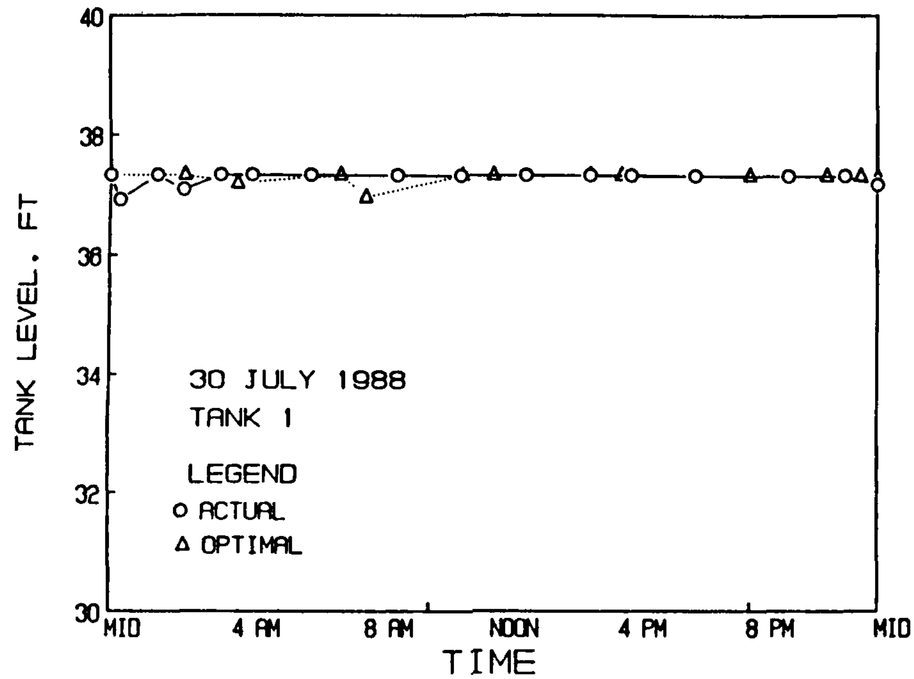

Figure 7. Tank 1 water levels for 30 July 1988.

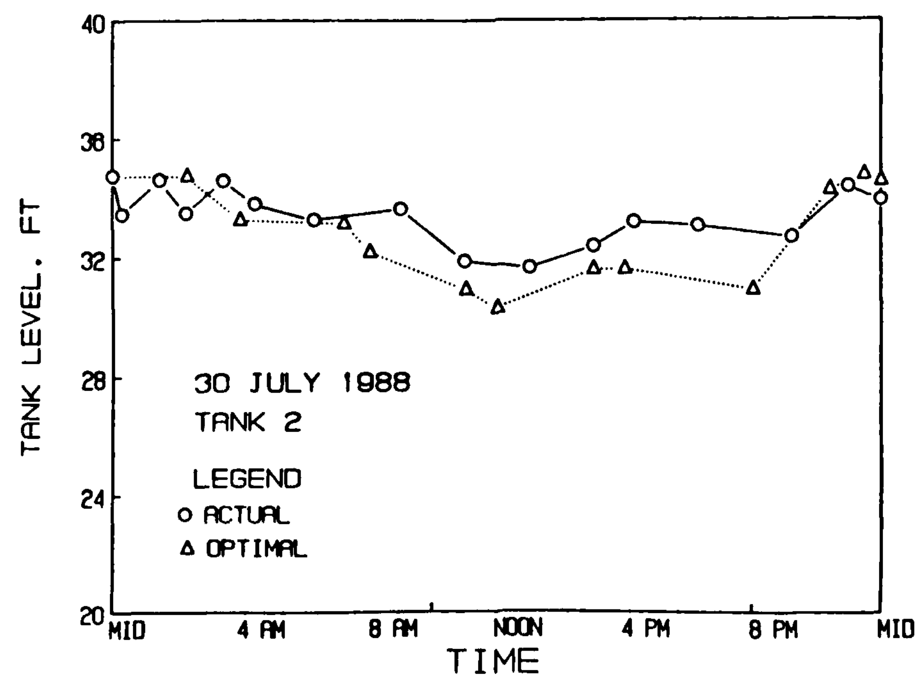

Figure 8. Tank 2 water levels for 30 July 1988.

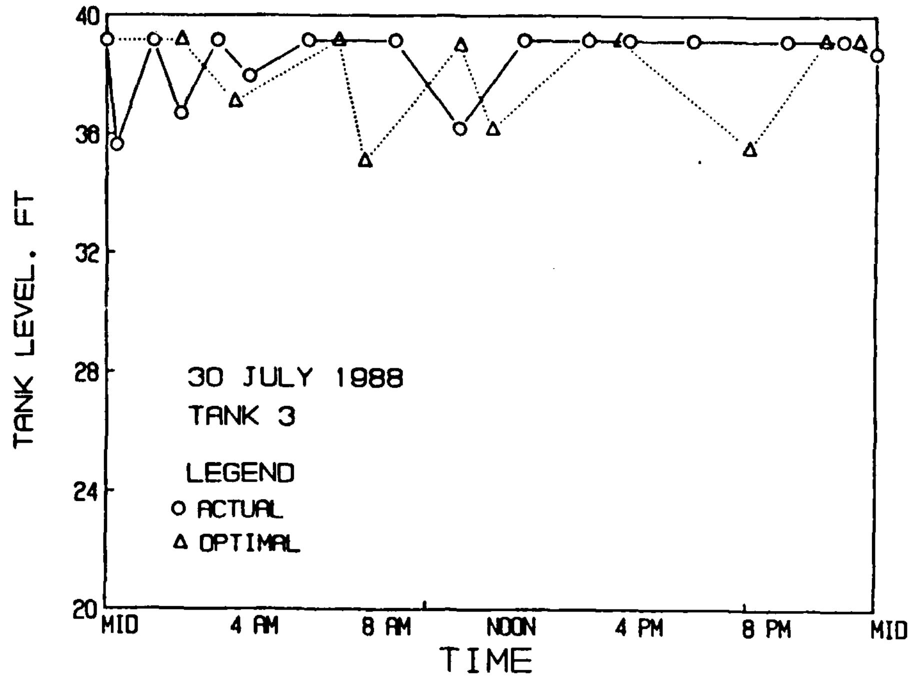

Figure 9. Tank 3 water levels for 30 July 1988.

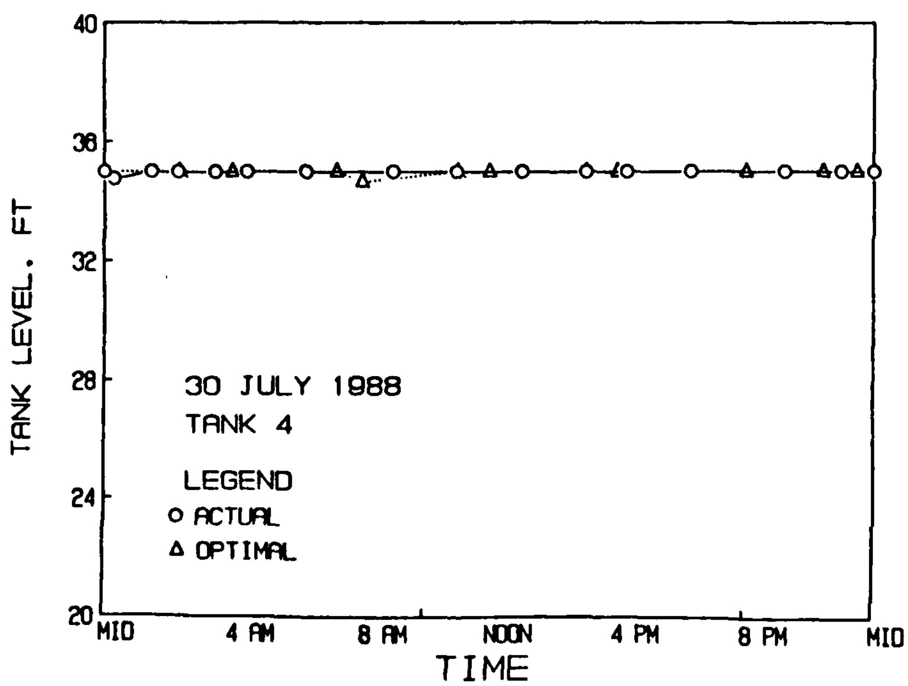

Figure 10. Tank 4 water levels for 30 July 1988.

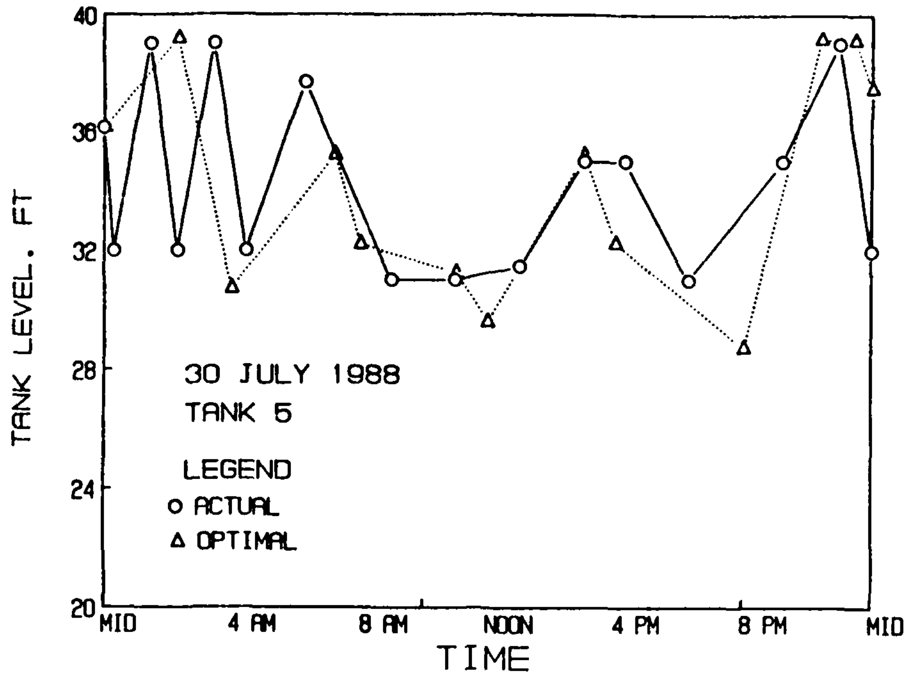

Figure 11. Tank 5 water levels for 30 July 1988.

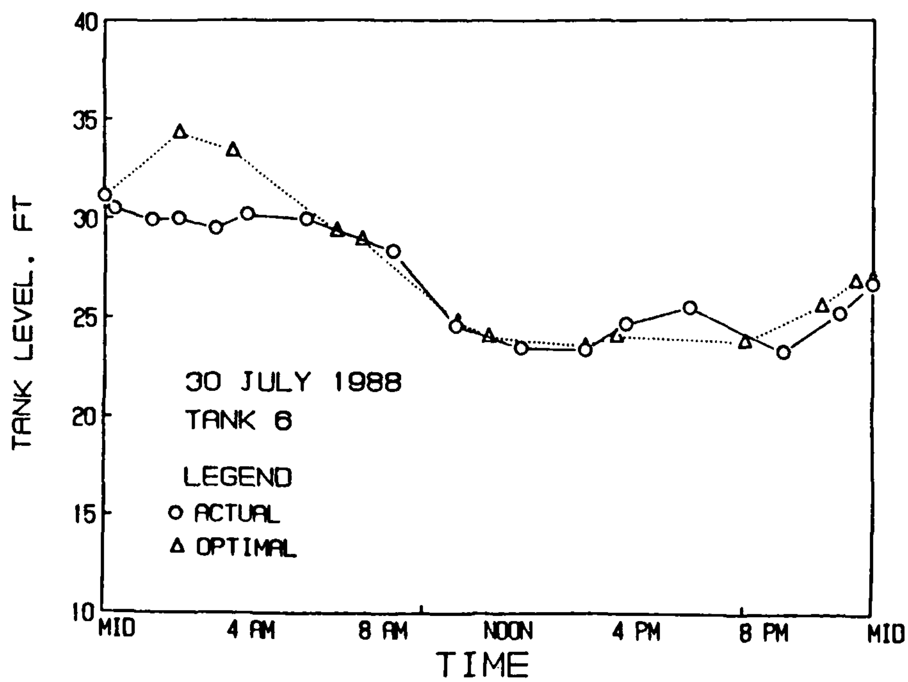

Figure 12. Tank 6 water levels for 30 July 1988.

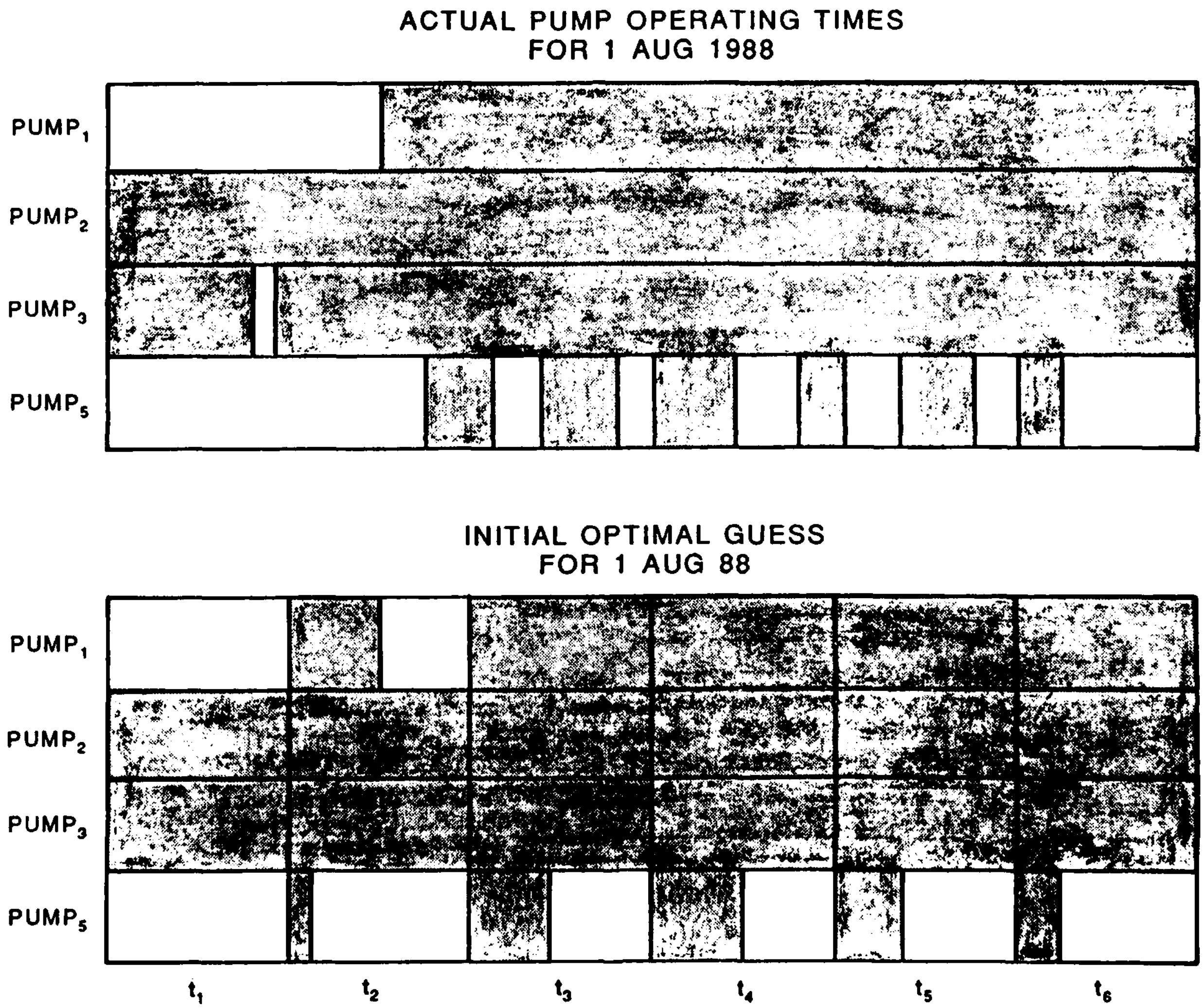

Figure 13. Actual and initial guess operating times for 1 August 1988.

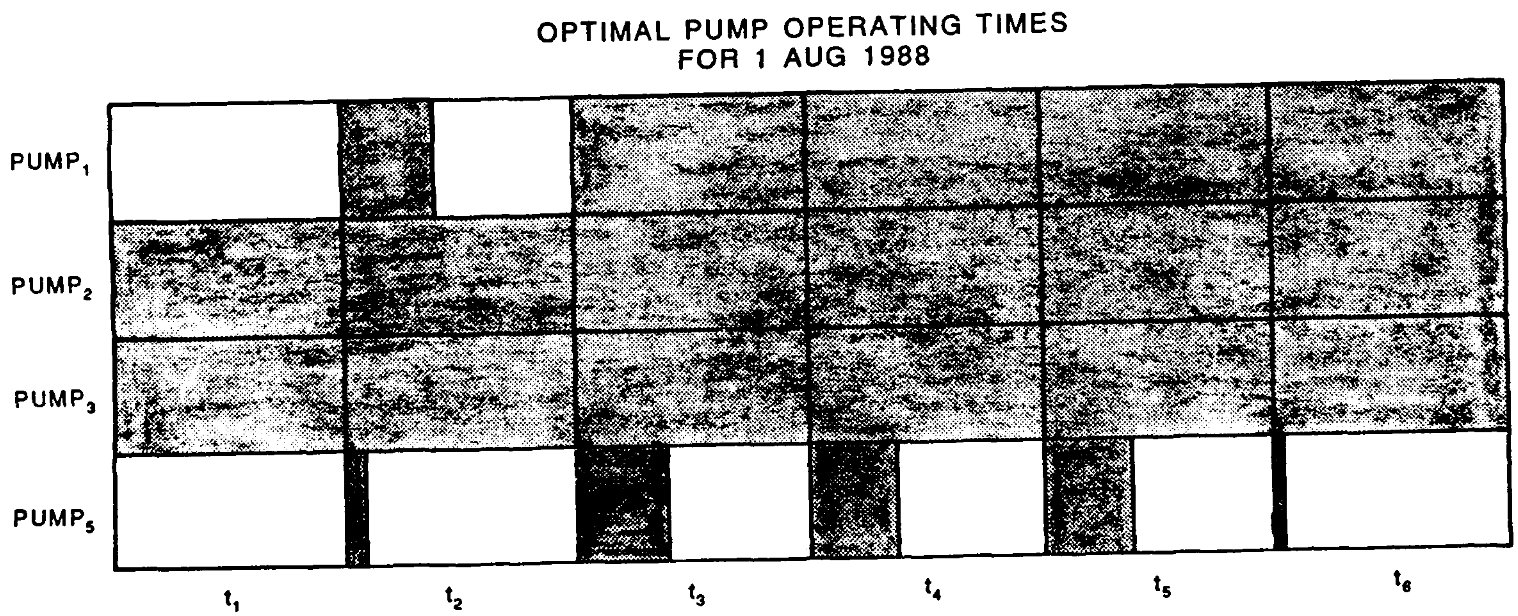

Figure 14. Optimal pump operating times for 1 August 1988.

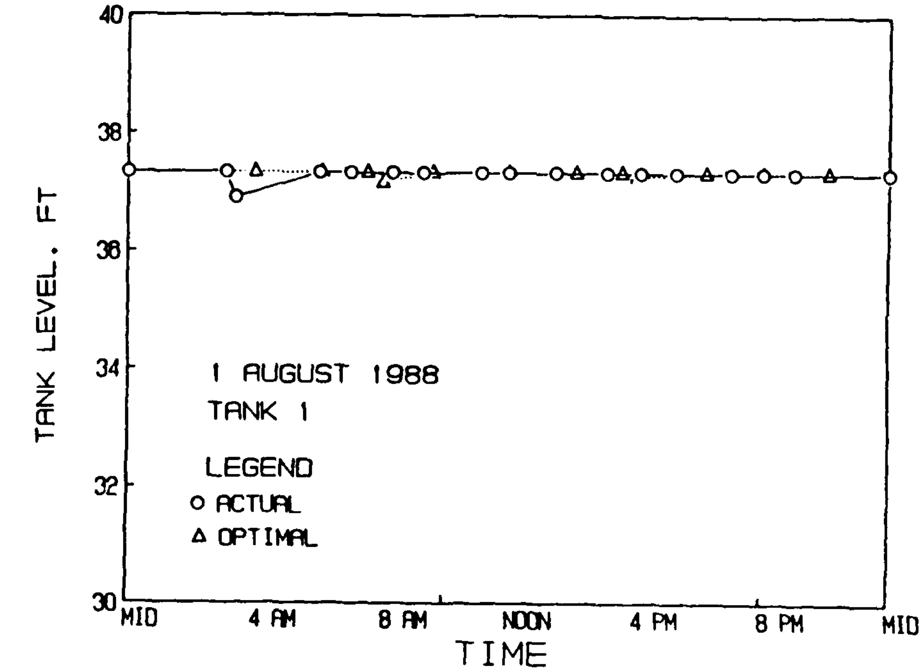

Figure 15. Tank 1 water levels for 1 August 1988.

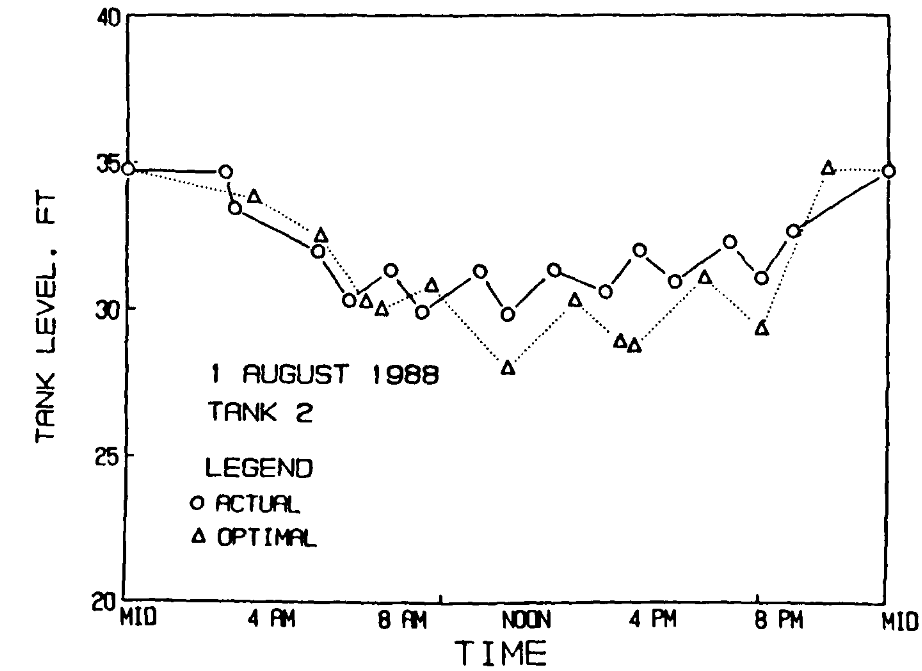

Figure 16. Tank 2 water levels for 1 August 1988.

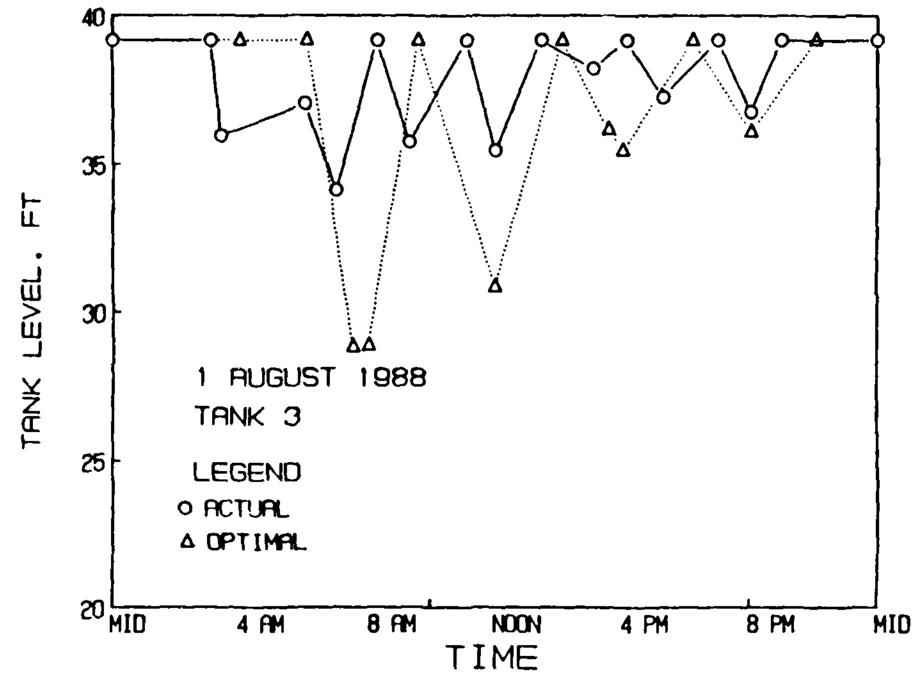

Figure 17. Tank 3 water levels for 1 August 1988.

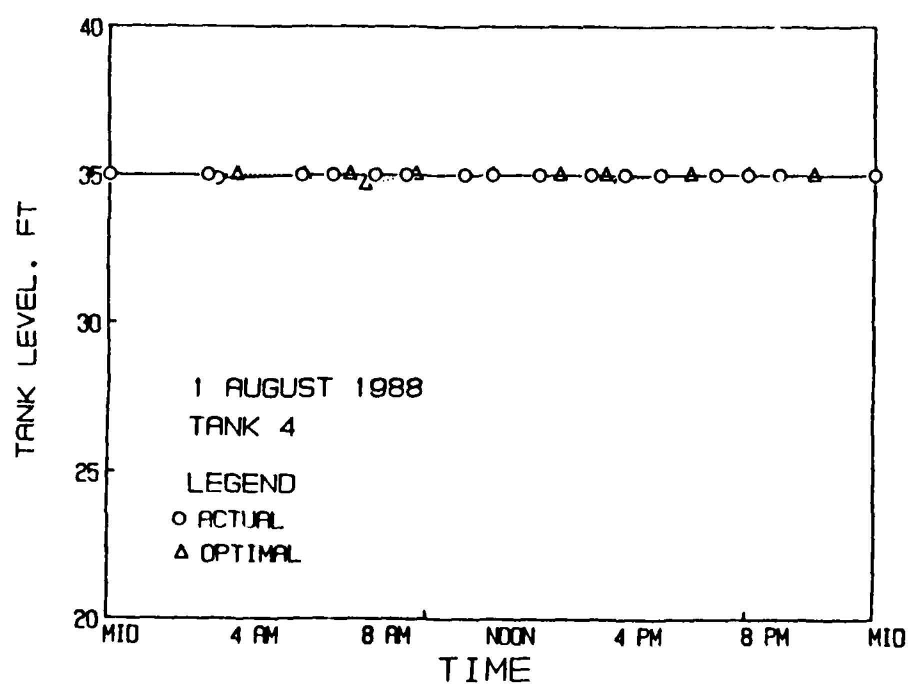

Figure 18. Tank 4 water levels for 1 August 1988.

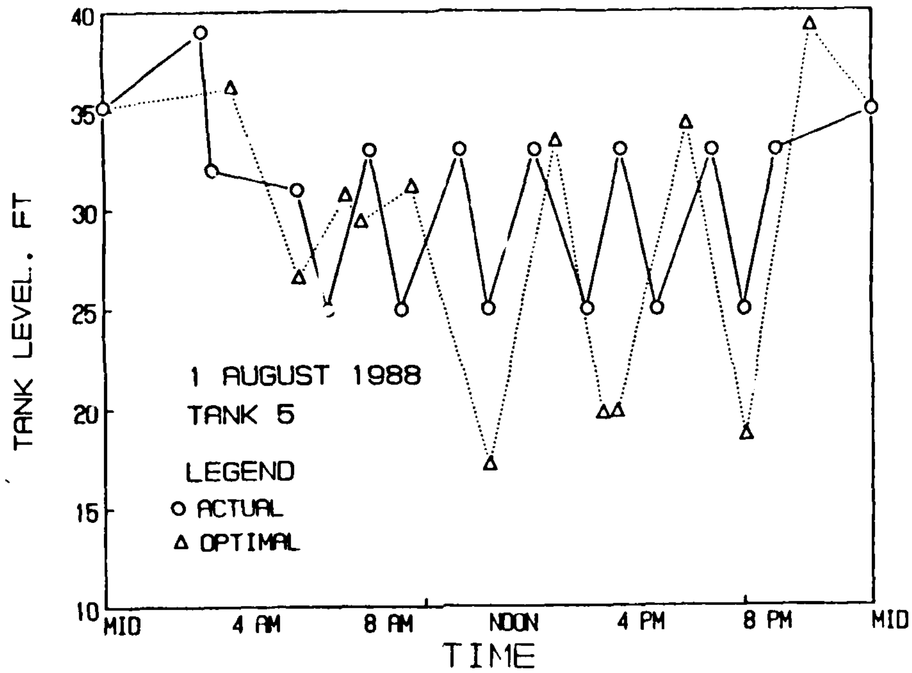

Figure 19. Tank 5 water levels for 1 August 1988.

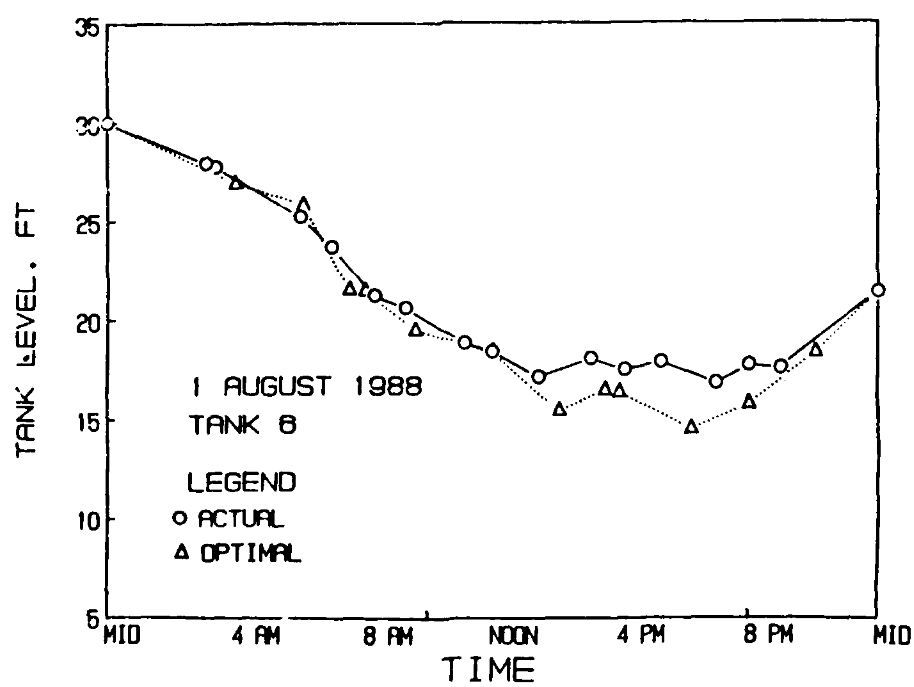

Figure 20. Tank 6 water levels for 1 August 1988.

Although the OCOPS results did not show an improvement over the estimated actual cost of operation, some comments can be made on the model as applied to Fort Hood. Several initial guesses for pump operating time were used during the study. These starting points ranged from ones with all pumps running for the entire day to those with only two pumps running for a portion of the day. In all cases, OCOPS converged to a solution within a few dollars--well within the accuracy of the simulation--of the estimated actual cost. This result is encouraging since the model was able to reach the same solution, regardless of the initial guess supplied.

Several reasons could explain why improvements to the estimated actual cost were not shown for the Fort Hood system. The use of a questionably calibrated mathematical model of the Fort Hood system almost certainly influenced the results. Another reason could be that the Fort Hood system may be so highly constrained and have such a narrow feasible path that the solution has been reached heuristically by trial and error. Although the system has 5 MG of elevated storage, it is distributed throughout the system. There may not be enough available storage in any one tank to provide the system with the flexibility needed for improved pump operation. Rather, the storage may be needed to supply system demands immediately instead of storing water for future use. Finally, the Fort Hood system may be very sensitive to the time pumps are operating If so, it may have been more realistic to use a smaller time interval than the 4-hr time step used in order to improve actual operation. Reducing the time interval would have reduced the constraint placed on time and provided flexibility by more closely approximating continuous operation.

Summary

The results from OCOPS were initially surprising since the cost provided by the model was expected to be less than the estimated actual cost of operation. However, due to the questionable calibration, it is unfair to speculate on the model's capabilities without first testing it on a well calibrated system for which real actual costs are known. It is possible that the Fort Hood system has been optimized heuristically. If so, then the results of OCOPS are indeed encouraging since they are very close to the "optimal" results. Before extolling the virtues of the model, however, OCOPS should be tested using a system with a truly calibrated simulation model. The results for the Fort Hood system must be treated with caution since the calibration of the water distribution system was questionable. To use OCOPS or any pump optimization model with confidence, the model must be able to simulate the response of the water distribution system accurately. If the modeled response of the system is not accurate, then the results of the optimization model may be misleading.

Although the results of this study indicate that costs provided by OCOPS are no less than the estimated actual cost of operation at Fort Hood, the technique appears to work. It is possible that room for improvement exists at Fort Hood, but without a well calibrated model, the magnitude of improvement is impossible to quantify.

Advantages of Pump Optimization Models

In general, water distribution systems are highly dimensional with multiple storage tanks and pressure zones. Most water systems have adequate storage to meet peak demands and provide reserve water for emergencies or fire fighting. However, in some cases, these same systems may be highly constrained. The storage available in these systems may be insufficient to store elevated water for use at later times.

A good optimization model capable of handling multiple storage tanks, coupled with a well calibrated model of the water distribution system, can identify the potential for improved operation. Such

improved operation cannot only surface in the form of reduced electrical costs, but also in more efficient system operation since these models generally have reliability constraints embedded within them.

It should be emphasized that, for some systems, a small percentage reduction in energy costs can translate into a large dollar savings. A pump optimization model can identify pump combinations that result in minimal energy costs. It is possible that some systems may be operating optimally without the use of a model. However, in these cases, an optimization model can still provide important information on how the system reacts to changes in system operation, storage, demand, electrical rate structure, and piping configuration.

5 CONCLUSIONS AND RECOMMENDATIONS

An optimization methodology to minimize pumping costs has been developed and applied to the Fort Hood, TX water distribution system. The methodology combines a nonlinear optimization algorithm, GRG2, with a water distribution simulation routine, WADISO. The two programs are linked together by a GCOMP routine which computes the values of the objective function and the problem constraints. The combined subroutines, along with a data entry routine, are contained in a computer program titled Optimal Control of Pump Stations (OCOPS).

OCOPS adjusts the fraction of time a pump operates during a particular interval to minimize electrical costs while maintaining system reliability. System reliability is described in the form of problem constraints on system storage and pressure. Additional constraints on the problem include explicit bound constraints on decision variables and implicit system constraints.

Implicit system constraints are the equations describing the head and flow distribution in a hydraulic network and are solved by the WADISO program. The remaining constraints are solved by GRG2. In OCOPS, the implicit bound constraints on system parameters include an upper and lower bound on nodal pressures and a maximum and minimum allowable tank level. Appendix A provides a detailed description of the problem formulation.

The optimization technique was applied to the Fort Hood water distribution system to test its capabilities. The model was applied to 2 days for which actual operating conditions were known: 30 July 1988 and 1 August 1988. The actual cost of pumping for 30 July 1988 was \$380.36, whereas the cost of pumping provided by the model was \$390.00. For 1 August 1988, the actual power cost was \$560.75 and the cost provided by the model was \$562.88. All problem constraints were satisfied under the final solution.

OCOPS was unable to improve the current pumping schedule for the Fort Hood system. Most likely this is due to a poorly calibrated model of the Fort Hood system. It is possible, however, that the Fort Hood system is so highly constrained with respect to system storage that it may already be operating optimally.

Even though OCOPS was unable to provide a pump schedule for Fort Hood which resulted in operating costs less than the estimated actual costs, the model should not be abandoned. Rather, the program should be tested on a system for which a well calibrated mathematical model is available to test the model's true capabilities. The model does appear to work since it converged to a solution very close to the estimated actual cost for a variety of initial pump schedules supplied.

A useful feature of OCOPS is its ability to handle systems with multiple storage tanks. Previous methods have generally relied on dynamic programming as the mathematical optimization technique. Although dynamic programming is very powerful, efficiency deteriorates rapidly with an increase in the number of storage tanks. Since OCOPS has a water distribution simulation routine embedded within it, the number of storage tanks that can be included in the optimization is limited by WADISO.

Some previous methodologies have used simplified representations or approximations of the distribution system hydraulics. In these approaches, it is possible for error to be introduced into the optimization process. In OCOPS, the governing equations of pressure and flow distribution are solved as necessary and no simplifying assumptions are made. This approach allows the optimization model to simulate extreme operating conditions.

All constraints in the optimal control problem formulated in this study are assumed to be binding and cannot be violated. Other techniques have incorporated penalty functions in the constraint set. Under

the latter approach, constraints can be violated. However, binding constraints prohibit constraint violation. Thus, the need for a well calibrated model of the distribution system is readily apparent since OCOPS initially satisfies all constraints and then attempts to optimize the system.

OCOPS is very flexible. Proposed changes to the distribution system can be analyzed by simply modifying the computer model of the network. As a result, the impact of a change to the system can be quantified and used to help decide if the change should be made.

OCOPS is written in FORTRAN-77 and can be run on any IBM-compatible PC with 640K of memory. The 1 August 1988 simulation took nearly 8 hr on a 80386-based computer. Proposed measures to reduce the amount of computation time are listed in Appendix A.

Most computer time is spent computing the partial derivatives of the objective function and of the problem constraints. A finite difference approximation is made, requiring a number of WADISO calls for each decision variable analyzed. By computing the derivatives analytically, computer run time could be significantly reduced. Furthermore, analytically computed derivatives eliminate any error introduced by the finite difference approximation. This, in turn, may cause the optimization algorithm to move closer to a global optimum.

The OCOPS model should be reapplied to a well calibrated system to fully determine the model's capabilities. A well calibrated model of a water distribution system can be created using high-quality data on system tank levels, demands, and layout.

Research is needed on analytically computing the derivatives of the objective function and constraints to reduce computation times. The OCOPS model can be run on a PC, but the program requires a large number of computations. As noted above, the number of computations could be reduced substantially by computing the derivatives analytically. Only in this way can the model truly approach real-time operation.

OCOPS should be modified to account for electricity demand charges. At present, the program considers only electricity unit charges. For some water utilities, electricity demand charges comprise the major fraction of electrical costs. These additional costs would be incorporated in the model through additional constraints.

After these modifications are made and validated, the model can be combined with a demand forecasting model and integrated into a Supervisory Control and Data Acquisition (SCADA) system to provide automated optimal control of water supply pump stations. The end result will be efficient operation, reduced electrical costs, and wise use of limited natural resources.

METRIC CONVERSION TABLE

1 hp	=	0.75 kW
1 gal	=	3.785 L
1 ft	=	0.305 m
1 psi	=	6.895 kPa

CITED REFERENCES

Aldworth, G. A., "Energy-Saving Pump Selection," *Journal of the American Water Works Association (JAWWA)*, Vol 75, No. 10 (1983).

American Water Works Association, Computer Assisted Design of Water Systems Committee, "Network Analysis Survey 1984," *Proceedings of the AWWA Annual Conference*, Dallas, TX (1984).

Aptowicz et al., "Using Elevated Storage and Off-Peak Pumping to Control Energy Costs," *JAWWA*, Vol 79, No. 11 (1987).

Army Regulation (AR) 11-27, *The Army Energy Plan* (July 1989).

Brailey, D., and A. Jacobs, "Energy Management in the Water Works Industry," *Journal of the New England Water Works Association*, Vol 94, No. 3 (1980).

Cembrano, G., M. Brdys, J. Quevedo, B. Coulbeck, and C. H. Orr, "Optimization of a Multi-Reservoir Water Network Using a Conjugate Gradient Technique. A Case Study," *8th International Conference on Analysis and Optimization of Systems* (Institut National de Recherche en Informatique et en Automatique, Antibes, France, 1988).

Chao, J., "Can Off-Peak Pumping Cut Utility Power Costs," *JAWAA*, Vol 71, No. 5 (1979).

Clark, T., "Reducing Power Costs for Pumping Water," *Opflow*, Vol 13, No. 10 (1987).

Clingenpeel, W. H., "Optimizing Pump Operating Costs," *JAWWA*, Vol 75, No. 10 (1983). 502-509.

Cohen, G., "Optimal Control of Water Supply Networks," *Optimization and Control of Dynamic Operational Research Models*, S. G. Tzafetas (Ed.) (North Holland Publishing, 1982).

Coulbeck, B., and M. J. H. Sterling, "Optimized Control of Water Distribution Systems," *Proceedings of IEE*, Vol 125, No. 9 (Institute of Electronic Engineers [IEE], 1978).

Coulbeck, B., "Optimal Operations in Non-Linear Water Networks," *Optimal Control Applications and Methods*, Vol 1 (1980).

Coulbeck, B., and C. H. Orr, "Optimized Pump Scheduling for Water Supply Systems," *Proceedings of the 3rd IFAC Symposium on Control of Distributed Parameter Systems*, Toulouse, France (1982).

Coulbeck, B., and C. H. Orr, "Optimized Pumping in Water Supply Systems," *Proceedings of IFAC, IX Triennial World Congress*, Budapest, Hungary, Vol 6, (1984).

Dreyfus, D. A., "Persistent National Energy Issue," *ASCE Journal of Professional Issues in Engineering*, Vol 114, No. 1 (American Society of Civil Engineers [ASCE], 1988).

Fallside, F., and P. F. Perry, "Hierarchical Optimization of a Water-Supply Network," *Proceedings of IEE*, Vol 122, No. 2 (1975).

Gessler, J., and T. M. Walski, *Water Distribution System Optimization*, Technical Report EL-85-11 (U.S. Army Engineer Waterways Experiment Station [USAWES], 1985).

Lackowitz, G. W., and P. J. Petretti, "Improving Energy Efficiency Through Computer Modeling," *JAWWA*, Vol 75, No. 10 (1983).

Lansey, K. E., *Optimal Design of Large Scale Water Distribution Systems Under Multiple Loading Conditions*, Ph.D. Dissertation (University of Texas, Austin, 1987).

Lasdon, L. S., and A. D. Waren, *GRG2 User's Guide* (University of Texas, Austin, 1986).

Ormsbee, L., and D. Contractor, "Optimization of Hydraulic Networks," *International Symposium on Urban Hydrology, Hydraulics, and Sediment Control*, Lexington, KY (1981).

Ormsbee, L. E., T. M. Walski, D. V. Chase, and W. W. Sharp, *Techniques for Improving Energy Efficiency at Water Supply Pumping Stations*," Technical Report EL-87-16 (USAWES, 1987).

Patton, J. L., and M. B. Horsley, "Curbing the Distribution Energy Appetite," *JAWWA*, Vol 77, No. 6 (1980).

Reheis H. F., and M. K. Griffin, "Energy Cost Reduction Through Operational Practices," *Proceedings of the AWWA Annual Convention*, Dallas, TX (AWWA, 1984).

Sabet, M. H., and O. J. Helweg, "Cost Effective Operation of Urban Water Supply System Using Dynamic Programming," *Water Resources Bulletin*, Vol 21, No. 1 (1985).

Shamir, U., "Computer Applications for Real-Time Operation of Water Distribution Systems," *Proceedings of the ASCE Specialty Conference on Computer Applications in Water Resources*, H. C. Torno (Ed.) (ASCE, 1985).

Sharp, W. W., and D. V. Chase, "Verifying Water Distribution Computer Model Calibration Results," *AWWA Computers and Automation Specialty Conference*, Denver, CO (AWWA, 1989).

Solanas, J. L., and M. Verges, "Extension of the Dynamic Programming Successive Approximation and Its Application to Automatic Operational Control of Water Distribution Systems," *IFAC-IFORS Symposium*, Varna, Bulgaria (1974).

Sterling, M. J. H., and B. Coulbeck, "Optimization of Water Pumping Costs by Hierarchical Methods," *Proceedings of the Institute of Civil Engineers*, Vol 59 (1975).

Sterling, M. J. H., and B. Coulbeck, "A Dynamic Programming Solution of Optimization of Pumping Costs, *Proceedings of the Institute of Civil Engineers*, Vol 59 (1975).

Tarquin, A. J., and J. Dowdy, "Optimal Pump Operation in Water Distribution," *ASCE Journal of Hydraulic Engineering*, Vol 115, No. 2 (1989).

Walski, T. M., *Analysis of Water Distribution Systems* (Van Nostrand Reinhold, 1984).

UNCITED REFERENCES

Coulbeck, B., "Optimization of Water Networks," *Transactions of the Institute of Measurement and Control*, Vol 6, No. 5 (1984).

Jeppson, R. W., *Analysis of Flow in Pipe Networks* (Ann Arbor Science, 1976).

Lasdon, L. S., R. L. Fox, and M. W. Ratner, "Nonlinear Optimization Using the Generalized Reduced Gradient Method," *Revue Francaise d'Automatique, Informatique et Recherche Operationnelle* (November 1974).

Lasdon, L. S., A. D. Waren, A. Jain, and M. Ratner, "Design and Testing of a Generalized Reduced Gradient Code for Nonlinear Programming," *ACM Transactions on Mathematical Software*, Vol 4, No. 1 (1978).

Ormsbee, L. E., "Optimal Control Methodologies for Water Distribution Systems," *Proposal Submitted to National Science Foundation* (Department of Civil Engineering, University of Kentucky, Lexington, 1988).

Shamir, U., "Optimal Design and Operation of Water Distribution Systems," *Water Resources Research*, Vol 10, No. 1 (1974).

Shamir, U., "Real-Time Control of Water Supply Systems," *Proceedings of the International Symposium on Real-Time Operation of Hydrosystems* (University of Waterloo, Canada, 1981).

Tatjewski, P., "A Suboptimal Approach To Scheduling of Reservoir Levels for a Multireservoir Water Distribution Network," *International Conference, Computer Applications for Water Supply and Distribution*, Leiscester, Great Britain (1987).

Wood, D. J., *Algorithms for Pipe Network Analysis and Their Reliability*, Research Report No. 127 (University of Kentucky, Water Resources Research Institute, Lexington, 1981).

APPENDIX A:

SOLUTION METHODOLOGY

Problem Formulation

Objective Function

The optimal control problem can be formulated mathematically as a nonlinear optimization problem subject to a series of constraints. In the approach presented in this report, pump operating time is treated as the decision variable. The operating period, usually 24 hr, is divided into a series of T time intervals. For a water distribution system with I pumps, the objective function of the optimal control problem can be written as:

$$\text{Minimize} \sum_{t=1}^{T} \sum_{i=1}^{I} F(X_{t,i}) \qquad \text{[Eq A1]}$$

where $X_{t,i}$ is the duration of time pump i operates during time interval t. $F(X_{t,i})$ is the total cost of pumping for pump i during time interval t.

Figure A1 shows a typical set of decision variables for a given 24 hr period. In this case, the operating period is divided into six time intervals of 4 hr each. If a pump operates during a time interval, it is assumed to begin pumping at the start of the time interval and operate continuously for X hr. For instance, during the hours between 8:00 a.m. and 12:00 p.m., pump 2 runs for 2.5 hr from 8:00 a.m. to 10:30 a.m.; thus, $X_{3,2} = 2.5$.

Problem Constraints

The nonlinear optimization problem is subject to a series of problem constraints. These constraints can be categorized as (1) explicit bound constraints on the decision variables, (2) implicit bound constraints on system parameters, and (3) implicit system constraints.

The *explicit bound constraints* set lower and upper limits on the values of the decision variables. For the optimal control problem, the explicit bound constraints can be expressed as:

$$0 \leq X_{t,i} \leq \Delta t \qquad \text{[Eq A2]}$$

where Δt is equal to time interval t . This constraint restricts the time a pump can operate from 0 to Δt hr during a given time interval; however, a pump can operate during any or all time intervals.

The *implicit bound constraints* set limits on system parameters such as pressure, flow rate, system storage, or total energy consumption for a given time interval. For any time interval, the system pressure at any node may be bound between a minimum acceptable pressure and a maximum allowable pressure. The implicit pressure constraint is expressed as:

$$Pmin_{k,t} \leq P_{k,t} \leq Pmax_{k,t} \qquad \text{[Eq A3]}$$

where $P_{k,t}$ is the pressure at node k during time interval t. High pressures (> 100 psi) are undesirable

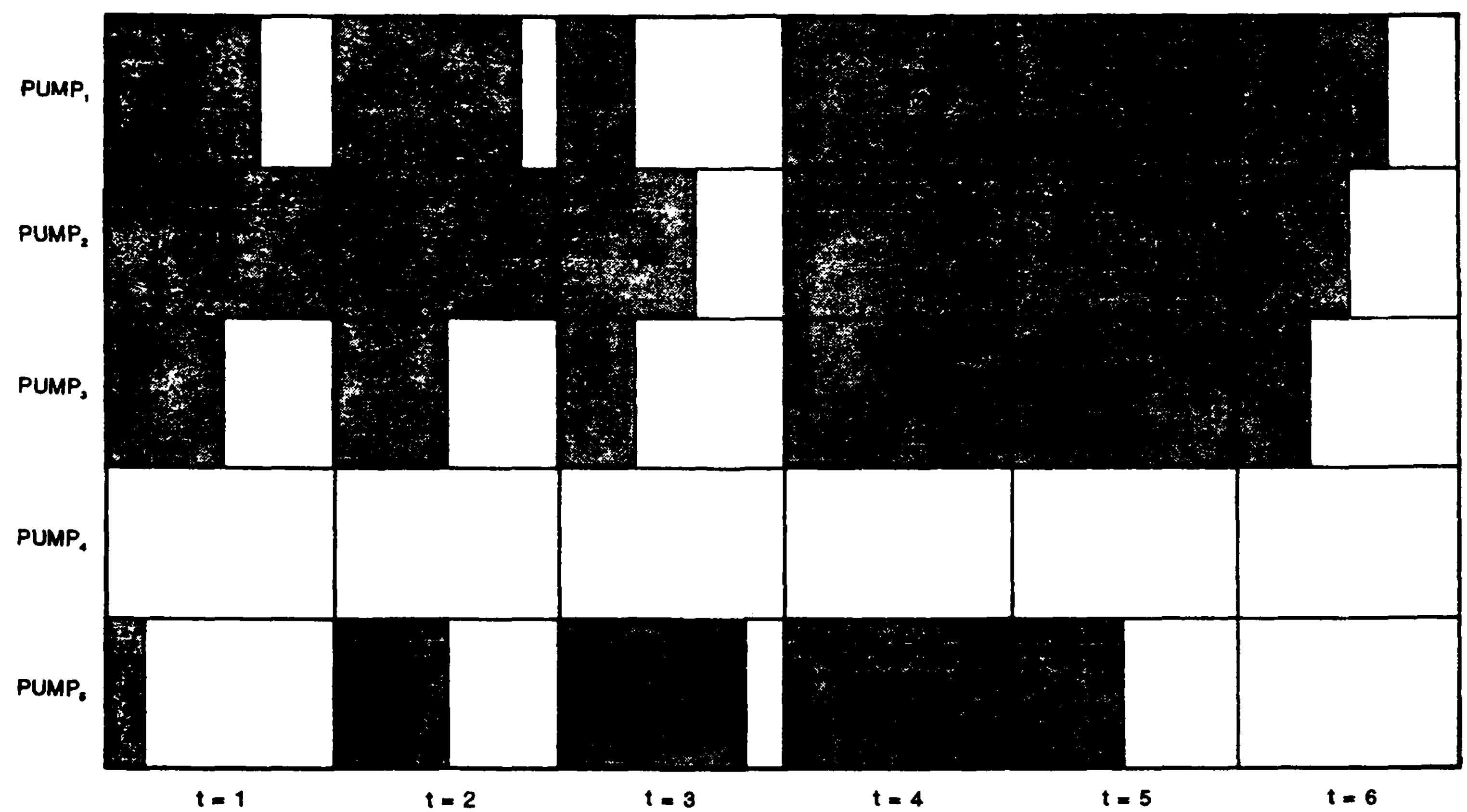

Figure A1. Representation of the decision variables. Shaded areas indicate pump is running.

since they have a tendency to waste water and could damage residential and commercial plumbing. Low pressures (< 40 psi) do not allow water to be delivered to building floors above four stories. Furthermore, many health codes require that pressures not fall below 20 psi. Pressures may, however, fluctuate between Pmin and Pmax during any time interval.

The flow rate through any given pipe p may also be bound between an acceptable minimum and allowable maximum during a given time interval. This constraint can be written as:

$$Qmin_{p,t} \leq Q_{p,t} \leq Qmax_{p,t} \qquad \text{[Eq A4]}$$

where $Q_{p,t}$ is the flow rate in pipe p during time interval t. Flow rates can be bounded to avoid excessive head loss in a line. For this study, this constraint was not included in the problem formulation.

Among the more important implicit system constraints are the bounds placed on tank water level. The level in tank m can be bound between a minimum and a maximum level. The tank level constraint can be expressed as:

$$Lmin_{m,t} \leq L_{m,t} \leq Lmax_{m,t} \qquad \text{[Eq A5]}$$

where $L_{m,t}$ is the water level in tank m during time period t. In most cases, the minimum allowable tank level, $Lmin_{m,t}$, will not be the actual bottom of the tank. Rather, $Lmin_{m,t}$ will be several feet above the bottom of the tank to allow for a reserve of water for emergencies, including fire fighting. Tank levels can vary between Lmin and Lmax during any time interval. It is the energy stored in the tanks that allows pump operation to be scheduled in such a way as to effect least-cost pumping.

The total electrical consumption of the pump station during any time interval may be bounded between a lower and upper value. This enables control of kilowatt consumption so that electrical demand charges can be minimized. The constraint for electrical consumption can be written as:

$$Emin_t \leq E_t \leq Emax_t \qquad \text{[Eq A6]}$$

where E_t is the combined electrical consumption of all pumps subject to the demand charge during time interval t. Kilowatt consumption can vary between Emin and Emax during a time interval.

The *implicit system constraints* represent the equations of conservation of mass and energy associated with hydraulic networks, and describe the state of the system. The nodal conservation of mass constraint requires that the sum of pipe flows into or out of a node, less any external or internal demands, be equal to zero. A conservation of mass constraint is necessary for each node and can be expressed as:

$$F_k(Q) = \sum_{n=1}^{N_k} Q_n - D_k = 0 \qquad k=1,2,3,\ldots K \qquad \text{[Eq A7]}$$

where Q_n is the flow rate in pipe n, D_k is the demand at node k, N_k is the number of pipes connected at node k, and K is the number of nodes in the system.

Since the flow in a pipe is a function of the head difference between the beginning and end of the pipe, Equation A7 can be rewritten in terms of head difference. This essentially combines the

conservation of mass and energy equations into a single equation. The advantage of combining these equations is a reduced number of simultaneous equations that must be solved. When using the Hazen-Williams head loss formula under steady-state conditions, the combined equation can be expressed as:

$$F_k(Q) = \sum_{n=1}^{N_k} \left[\frac{(H_B - H_E)_n}{CP_n} \right]^{1/m} - D_k = 0 \qquad k=1,2,3,\ldots K \qquad \text{[Eq A8]}$$

where:

$$CP_n = \frac{4.73\ L_n}{C_n^{1.852}\ D_n^{4.87}}$$

H_B = Head at the beginning of pipe n, ft

H_E = Head at the end of pipe n, ft

L_n = Length of pipe n, ft

D_n = Diameter of pipe n, ft

C_n = Hazen-Williams roughness coefficient for pipe n

m = 1.852.

Optimization Methodology

To solve the optimization problem, the nonlinear objective or cost function must be minimized subject to explicit bound constraints, implicit bound constraints on system parameters, and implicit system constraints. This results in a very complex problem. The complexity of the problem can be reduced substantially by linking a nonlinear optimization algorithm with a water distribution simulation routine. The implicit system constraints are solved by the simulation routine while the optimization algorithm computes the decision variables.

That was the approach taken in the optimization methodology presented in this study. The nonlinear optimization algorithm used was GRG2. The simulation routine used was WADISO. The two programs were linked together using the GCOMP subroutine which computes the values of the objective function and the constraints. The combined algorithms, along with a main program, are contained in the computer program OCOPS.

The main program allows data entry and initiates execution of the nonlinear optimization algorithm. The required data include a WADISO system file and an optimization data file. The WADISO system file is the mathematical model of the distribution system and contains information such as pipe length, roughness, diameter, node demand, and elevation. Optimization data include information on the pumps and tanks in the system, the number of increments in the operating period, the initial starting point, electrical costs, and GRG2 tolerances. A complete listing of the data entry portion of the program is provided in Appendix B.

The GCOMP routine, which evaluates the objective function and problem constraints, is required by GRG2 to determine the reduced gradient, step size, and whether a solution is optimal. This routine also updates the tank levels. For the optimal control problem, the objective function is the total cost of pumping for a 24-hr period.

Accurately computing pumping costs over a 24-hr period is complicated because cost is a function of pump head, discharge, and efficiency. To further complicate matters, pump head and efficiency vary with discharge and pump discharge varies with system demands and tank water levels. Assuming electricity costs can also vary over time, the equation required to compute pumping costs for one pump can be represented by:

$$\text{Cost} = \frac{0.746\gamma}{550}\int_0^T H_t \, R_t \, \frac{Q_t}{\ell_t} \, dt \qquad \text{[Eq A9]}$$

where:

H_t = Pump head at time t, ft

Q_t = Pump discharge at time t, cu ft/sec

ℓt = Pump efficiency at time t

R_t = Electrical rate at time t, \$/kWh

T = Operating period, hr

γ = 62.4 lb/cu ft.

Pumping costs can be approximated by dividing the operating period T into several time intervals of t hr each and finding the average cost over each time interval. That is the approach taken by OCOPS. At the beginning of each time interval, the implicit system constraints are satisfied by WADISO and values for pump head and discharge are passed back to the GCOMP subroutine where the cost of pumping is computed. Flows in lines connecting tanks to the system are also passed to the GCOMP routine so that tank levels can be updated. At the end of the time interval, the system is rebalanced based on the updated tank levels. Again, pump head and discharge values are passed to GCOMP so that a cost of pumping corresponding to the new system conditions at the end of the time interval can be computed. The two pumping costs are averaged to find the cost of pumping for the time interval.

During any time interval, pumps can be taken offline or tanks can close full or empty. If this occurs, the time to such a change is found, the implicit system constraints are satisfied immediately before the change, values for pump head and discharge are passed to GCOMP, tank levels are updated, and an average cost for the subinterval is computed. The system is also balanced immediately after the change. Critical information is passed to GCOMP where it is used to compute the average cost for the subinterval corresponding to the next change or the end of the time interval. Tank levels are also updated for the next subinterval.

The mathematical form of the objective function for i number of pumps using several time intervals can be expressed as:

$$\text{Cost} = \sum_{t=i}^{T} \sum_{i=1}^{I} \frac{0.746\gamma}{550} H_{t,i} \frac{Q_{t,i}}{\ell_{t,i}} * R_t * X_{t,i} \qquad \text{[Eq A10]}$$

where:

$H_{t,i}$ = Pump head of pump i during time interval t, ft

$Q_{t,i}$ = Pump discharge of pump i during time interval t, cu ft/sec

$\ell_{t,i}$ = Efficiency of pump i during time interval t

R_t = Electrical rate during time interval t, \$/kWh

$X_{t,i}$ = Time pump i runs during time interval t, hr

γ = 62.4 lb/sq ft

T = Operating period, hr

I = Total number of pumps optimized.

In addition to computing the objective function, the GCOMP subroutine also computes the values of the problem constraints. In OCOPS, the problem constraints computed by GCOMP are pressure at critical nodes and the tank water levels. At the end of each time interval, the implicit system constraints are solved by WADISO. The head at each node and the flow in each line connected to a tank are passed back to the GCOMP routine. This information is used to find the pressure at critical nodes and to update tank water levels so that values for the problem constraints can be found. A flowchart of the GCOMP routine is shown in Figure A2 and a complete listing of the subroutine is provided in Appendix C.

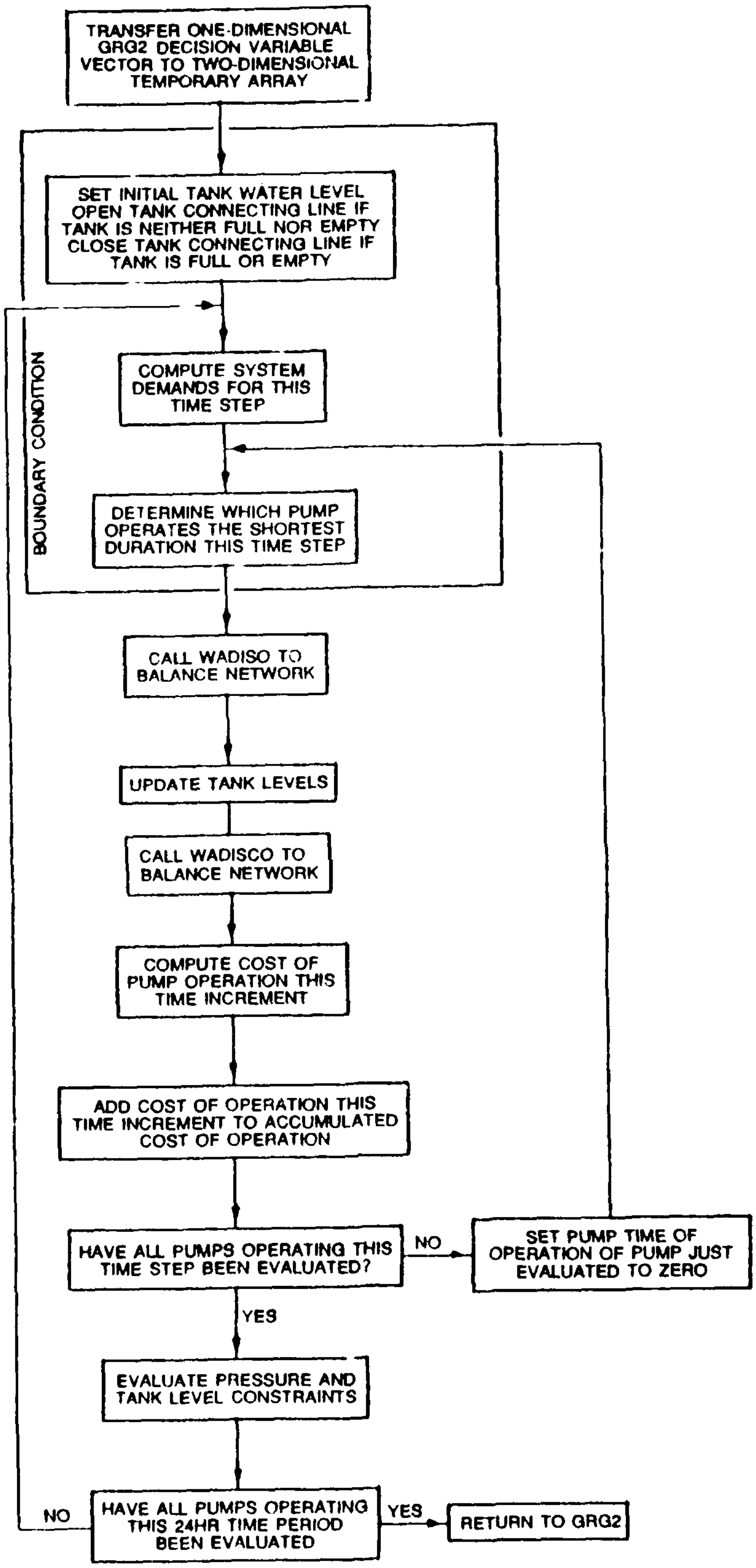

Figure A2. The GCOMP routine.

GRG2 Solution Algorithm

GRG2 is an optimization algorithm that solves nonlinear problems using a generalized reduced gradient technique. Problems are of the general form:

$$\text{Minimize} \quad F(X) \qquad \text{[Eq A11]}$$

Subject to:

$$LB_i \leq G_i(X) \leq UB_i \qquad i=1,2,3...m \qquad \text{[Eq A12]}$$

$$LB_j \leq X_j \leq UB_j \qquad j=1,2,3...n \qquad \text{[Eq A13]}$$

X is a one-dimensional vector with n elements called decision variables. F(X) is the objective function that is minimized subject to the set of problem constraints, $G_i(X)$. There are m linear or nonlinear constraints to the problem, each with a lower bound, LB_i, and an upper bound, UB_i. It is assumed that there are more decision variables than problem constraints; otherwise, the problem would be infeasible or have a unique solution. There are also lower and upper bounds on the decision variables.

The underlying theory of GRG2 is to find a solution for m of the variables, or basic variables, in terms of the remaining n-m variables, or nonbasic variables. This is accomplished through rewriting the constraint set as equality constraints by adding slack variables. The new constraints written in terms of basic and nonbasic variables are expressed as:

$$g_i(y(x),x) = 0 \qquad i=1,2,3...m \qquad \text{[Eq A14]}$$

where:

y(x) = Vector of basic variables

x = Vector of nonbasic variables.

The nonlinear problem can now be transformed into a reduced problem expressed as:

$$\text{MIN} \qquad F(X) = f(y(x),x) \qquad \text{[Eq A15]}$$

subject only to lower and upper bounds on the nonbasic variables. GRG2 solves the original nonlinear problem by solving a series of reduced unconstrained minimization problems.

GRG2 can optimize problems if an infeasible starting point is provided. An infeasible starting point is one where at least one problem constraint is violated. In this case, the algorithm enters into a Phase 1 optimization where the objective function becomes the sum of the constraint violations. The Phase 1 objective function is minimized until all constraints are within their bounds. At this point, the Phase 2 optimization begins.

Once the reduced problem has been formulated and a feasible starting point obtained, GRG2 uses the following steps to minimize the objective function.

1. The basis inverse and reduced gradient are computed and used to test the optimality of the feasible starting point. Optimality is checked via the Kuhn-Tucker conditions or if the fractional change in the objective function is less than a specified tolerance for a specified number of iterations.

2. If the starting point is not optimal, a new decision vector (xnew) is generated using the following recursive equation:

$$x_{new_i} = x_{old_i} + \alpha\, d_i \qquad i = 1,2,3\ldots n \qquad \text{[Eq A16]}$$

where d_i is the search direction and α is the step size.

3. The search direction is computed by:

$$d_i = -H_i\, \nabla F(X_i) \qquad i = 1,2,3\ldots n \qquad \text{[Eq A17]}$$

where $-H_i$ is an n by n symmetric positive definite matrix and $\nabla F(X_i)$ is the reduced gradient.

4. The magnitude of α is determined by substituting Equation A16 into Equation A11 and determining the value of α that minimizes the resulting one-dimensional objective function shown below:

$$\text{MIN} \quad F\,(x_i + \alpha\, d_i) \qquad \text{[Eq A18]}$$

5. The optimal value of the step size, α, is determined using a one-dimensional line search. During the line search, constraints on the original nonlinear problem are checked for violation. If a constraint is violated, the step size will be adjusted to keep the search within the bounds of the feasible region. Once the optimal value for α is found, the new decision vector is determined using Equation A16 and the process is repeated.

Note: if a variable crosses an explicit bound, that variable is set to the value of the bound and a change of basis occurs. The change of basis results in the bounded variable leaving the basis and a new variable not at a bound entering the basis. At this point, optimality is checked. If the new point is not optimal, the process is repeated.

WADISO Solution Algorithm

WADISO[20] is a computer simulation model used to determine the pressure and flow distribution within a water distribution system. Flow and pressure within a pipe network can be determined by either of two methods: (1) the loop method or (2) the node method. The loop method initially solves for the flow in the pipes and backs out the nodal heads. In contrast, the node method solves for the heads at the nodes and then computes pipe flow. WADISO uses the node method to determine the pressure and flow distribution in a network. Both methods provide accurate results; however, the node method offers several advantages over the loop method in the optimal control application presented here.

[20] J. Gessler and T. M. Walski.

The number of equations that must be solved to obtain the pressure and flow distribution is generally less with the node method. This is especially true for large systems since the number of equations is equal to the number of nodes. In the loop method, the number of equations is equal to the number of pipes in the system. Therefore, computer storage and execution time can be reduced by using the node equations. This choice of method becomes very important in optimizing large pipe networks because the equations describing the water distribution system must be solved many times.

The loop method requires that the system geometry be determined so that an energy path between two known head sources can be found. Analysis of the system geometry is not necessary with the node method. Also, due to the nature of the node equations, pressure control devices such as pumps, pressure reducing valves, and check valves can be included in the computer model without any increase in computer time.

The node method is based on the equation of continuity which states that all mass arriving at a node (Q_{in}) must equal all mass leaving a node (Q_{out} + D). Fluid can reach a node as flow through other connecting pipes or as a system demand placed at the node. Continuity at a node is expressed as:

$$\sum_{n=1}^{N} Q_{n\ out} - \sum_{n=1}^{N} Q_{n\ in} = D \qquad \text{[Eq A19]}$$

The flow rate in a pipe is a function of the head difference between the beginning and end of a pipe as shown by the Hazen-Williams formula:

$$H_B - H_E = \frac{4.73\ L\ Q^{1.852}}{C^{1.852}\ D^{4.87}} = CP \times Q^m \qquad \text{[Eq A20]}$$

As a result, the equation of continuity can be rewritten in terms of the head drop across a pipe and the characteristic pipe coefficient as shown below:

$$\sum_{n=1}^{N} \left[\frac{(H_B - H_E)_n}{CP_n} \right]_{out}^{1/m} - \sum_{n=1}^{N} \left[\frac{(H_B - H_E)_n}{CP_n} \right]_{in}^{1/m} = D \qquad \text{[Eq A21]}$$

WADISO also can incorporate pumps in the system equations. A pump is considered to be a pipe with a head flow relationship equal to the pump curve. The head-flow characteristics of a pump can be closely approximated by a parabola of the form:

$$H_p = A + BQ + CQ^2 \qquad \text{[Eq A22]}$$

Using the quadratic rule to solve a second-order polynomial and further assuming that the linear and squared coefficients are negative, the discharge from a pump can be expressed as a function of head as shown in Equation A23. It is necessary that the linear term of the pump characteristic curve be negative since a positive value would imply a curve with a maximum head at a positive discharge. A negative squared term ensures that the curve is concave. Either of these conditions could cause convergence problems while solving the system of equations.

$$Q = \frac{-B \pm \sqrt{B^2 - 4AC}}{2A} \qquad \text{[Eq A23]}$$

If one equation expressing mass continuity is written for each node in the system, then a K by K (K equals the number of nodes in the system) coefficient matrix of node equations can be established. Fortunately, the coefficient matrix is symmetrical and sparse so that only half of the matrix is used to solve the equations, thereby reducing computer time. WADISO uses a Gaussian elimination technique to solve the equations simultaneously. Determining the pressure and flow distribution is an iterative process with the flow rates of the previous iteration updating the coefficient matrix of the current iteration.

General Considerations

The optimal control problem is very large and complex. The size of the problem and the computer time required to solve it can be reduced by adhering to several simple guidelines.

The number of decision variables is equal to the product of the number of pumps to be optimized and the number of time intervals in the 24-hr time period. The computer time necessary to solve the optimal control problem can be reduced significantly by reducing the number of decision variables. These variables can be reduced by including only those pumps run on a daily or weekly basis. In other words, any pumps that are usually on stand-by or used only in case of emergency are not included in the analysis. Also, pumps that use an alternative fuel, such as diesel fuel, should not be included in the analysis since the methodology assumes all pumps are driven by electricity.

Another way to reduce the size and complexity of the problem is to skeletalize the water distribution system. When using a computer model of a water distribution system, it is not necessary to include all pipes in the system. Rather, only the major pipelines are included. Results from the skeletal model are still accurate because those lines not included do not carry a large amount of flow.[21]

WADISO uses the nodal method to solve the equations of continuity and energy associated with water distribution systems. Therefore, by reducing the number of pipes in the mathematical model of the system, the number of nodes can be lessened and computation time significantly reduced.

In most distribution systems, pumps are placed online at any time during the day. In other words, pumps operate over a continuous timeframe. In the optimal control methodology presented here, pumps operate over a discrete timeframe since it is assumed that pumps are placed online at the beginning of a time interval. The smaller the time interval used, the closer the approximation to continuous operation.

Ideally, a very small time interval would be used since it affords more flexibility in pump operation. Unfortunately, the size of the time interval is inversely proportional to the computer run time and required computer storage. Computer run time can be reduced substantially by using a larger time step equal to 3 or 4 hr as opposed to 0.5 or 1 hr. However, increased savings in pump cost may be realized by using a smaller time interval.

[21]T. M. Walski.

GRG2 requires an initial guess to start the optimization process. The guess can be either feasible or infeasible. A feasible initial guess is one that does not violate any problem constraints and an infeasible guess is one that violates at least one problem constraint. GRG2 can optimize pump operation even if an infeasible starting point is provided; however, the time required to solve the problem is increased substantially by such a point.

Several simple checks can be made to determine if an infeasible starting point is used. If a pump operating time is less than zero or greater than the specified time interval, GRG2 will change the value of the decision variable to its lower or upper bound, respectively. Although this type of infeasible starting point has very little effect on run time, it is good practice to provide an initial time of operation between zero and the specified time interval.

Node pressures have been formulated as problem constraints. The problem size can be reduced by specifying only critical nodes as pressure constraints. In other words, only examine those nodes most likely to have pressures that violate an upper or lower boundary. Nodes at high or low elevations or those at the extreme end of the distribution system can be considered critical nodes.

If, for a given initial guess, the pressure constraint is violated, then a new initial guess that does not violate the constraint can be used. The new guess should reflect how and when the constraint was violated, and be based on knowledge of the system. For example, if the pressure at a node is below the acceptable minimum during time step 3, then a pump can be placed online during this or a previous time step.

Tank water level at the end of the 24-hr period has also been formulated as a problem constraint. This was done because it is desirable to have tank levels at a predetermined elevation at the end of the operating period. The problem has been formulated so that the specified elevation is equal to the lower bound of the constraint. The nature of the optimal control problem forces the tank levels toward their lower bound. As a result, the specified ending elevation is met.

If an initial guess provided is such that at least one tank level at the end of the operating period is less than the specified elevation, a problem constraint is violated. By turning on pumps or running pumps longer during the last time interval, the tank constraint may not be violated. However, GRG2 can optimize problems with an infeasible starting point.

APPENDIX B:

MAIN DATA ENTRY ROUTINE

The main data entry routine opens and reads information from two data files--one describing the water distribution system and another containing information on pump characteristics, electrical costs, etc. The file that describes the water distribution system is actually a WADISO data file created using the WADISO computer program. A WADISO system file is the mathematical representation or computer model of the water distribution system. Information contained in this file includes pipe length, diameter, roughness, node demand, and elevation. The pump head-discharge relationship is also contained in this file.

The second file opened in the main data entry routine defines the number of pumps and tanks in the system and the time interval to be used for the analysis. The relationship between pump efficiency and pump discharge is also contained in this file. Other information includes the initial starting point for the nonlinear optimization routine, the cost of electrical energy, tank geometry and connecting pipes, the global demand factor for each time increment, and GRG2 tolerances.

The main data entry routine also defines variables and initializes several arrays used by GRG2. Variables defined by this routine include the number of decision variables and the number of problem constraints. The arrays include the initial guess of the decision variables, the upper and lower bounds on the decision variable, and the upper and lower bounds on the problem constraints. The nonlinear optimization algorithm is called from the main data entry routine.

The specific format for the WADISO system file can be found in the WADISO user's guide.[22] The WADISO program and documentation can be obtained from the author. The format of the system optimization data file is described in Table B1 and a sample input file is shown in Figure B1. The sample data file assumes a 4-hr time interval with four pumps in the distribution system available for optimization. The source code for the main data entry routine is provided in Figure B2.

[22] J. Gessler and T. M. Walski.

Table B1

Format of System Optimization Data File

Card Number	Format	Column	Description	Variable Name
1	I5	1-5	Number of Pumps Available for Optimization	NPUMP
	I5	6-10	Time Step Used (Hr)	NSTEP
	I5	11-15	Number of Tanks in the System	NTANK
	I5	16-20	Print Flag	IBUG
			Pump Efficiency Card **One Card for Each Pump Optimized; I=1, NPUMP**	
2	I5	1-5	Link Number of Pump	LINK(I)
	F10.2	6-15	First Discharge Point	XØ
	F10.2	16-25	Efficiency Corresponding to First Discharge Point	YØ
	F10.2	26-35	Second Discharge Point	X1
	F10.2	36-45	Efficiency Corresponding to Second Discharge Point	Y1
	F10.2	46-55	Third Discharge Point	X2
	F10.2	56-65	Efficiency Corresponding to Third Discharge Point	Y2
			Initial Starting Point Card **One Card for Each Decision Variable; I=1, NVAR**	
3	F10.2	1-10	Initial Time of Operation for Pump I During Time Interval (Hr)	GUESS
	F10.2	11-20	Electrical Rate in Effect for This Pump During This Time Interval (Cents/kWh)	ERATE(I)
			Node Constraint Data Card	
4	I5	1-5	Number of Critical Nodes in the System	NNODES
	F10.2	6-15	Minimum Acceptable Pressure (psi)	PMIN
	F10.2	16-25	Maximum Allowable Pressure (psi)	PMAX
5	16I5	1-5	Node Number of Critical Node I=1, NNODES	NNNODE(I)

Table B1 (Cont'd)

Card Number	Format	Column	Description	Variable Name
			Tank Data Card	
			One Card for Each Tank; I=1, NTANK	
6	I5	1-5	Node Number of Tank	NNNODE(I)
	I5	6-10	Link Number of Tank Connecting Line	NCON(I)
	F10.2	11-20	Top Elevation of Tank (ft)	TOPEL(I)
	F10.2	21-30	Bottom Elevation of Tank (ft)	BOTEL(I)
	F10.2	31-40	Tank Diameter (ft)	DIAM
	F10.2	41-50	Tank Level Desired at the End of the 24-Hr Period (ft)	ENDEL(I)
	F10.2	51-60	Minimum Allowable Tank Level for Fire Storage (ft)	STOMIN(I)
			System Demand Data Card	
7	8F10.2	1-10	Global Demand Factor for This Time Interval I=1, (24/NSTEP)	RATIO(I)
			GRG2 Data Card No. 1	
8	A90	1-90	Title for This Simulation	TITLE
			GRG2 Data Card No. 2	
9	F15.14	1-15	A Constraint Is Assumed To Be Binding If It Is Within This Value (FPNEWT) of One of Its Bounds	FPNEWT
	F15.14	16-30	If It Is Desired to Run the Problem With FPNEWT Initially Set Fairly Large and Then Tightened at the End of the Optimization, This Is Accomplished by Assigning FPINIT the Initial Tolerance and FPNEWT the Final One	FPINT
	F15.14	31-45	Minimum Fractional Change in Objective Allowed To Avoid Program Termination	FPSTOP
	F10.2	46-55	Step Size for Finite Difference Approximation of Partial Derivatives of Constraints and Objective Function	PSTEP

Table B1 (Cont'd)

Card Number	Format	Column	Description	Variable Name
			GRG2 Data Card No. 3	
10	I5	1-5	If Fractional Change in Objective Function Is Less Than FPSTOP for NNSTOP Iterations, Then Program Terminates	NNSTOP
	I5	6-10	If Newton Routine Requires More Than IITLIM Iterations Before Converging, Then Iterations Are Stopped and Corrective Action Is Taken	IITLIM
	I5	11-15	Print Flag	IIPR
	I5	16-20	If IIPN# is Greater Than Zero, Then IPR Will Be Set to # After IIPN# Iterations	IIPN#
	I5	21-25	Method for Initial Estimates of Variables for Each One-Dimensional Search	IIQUADBasic
	I5	26-30	Method for Obtaining Partial Derivative of Constraints and Objective Function 0 - Forward Difference Approximation 1 - Central Difference Approximation 2 - User-Supplied Subroutine	LDERIV
	I5	31-35	Method for Determining Conjugate Gradient	MMODCG

```
4    4    6    1
7        800.          69.      1300.        83.      1800.        74.
8       1600.          71.      2300.        82.      3000.        75.
9       2000.          72.      3500.        85.      4700.        70.
11      4000.          78.      5500.        86.      7500.        80.
     0.00        4.50          1      7           1
     4.00        4.50                 8           2
     4.00        4.50                 9           3
     0.00        4.50                11           4
     1.60        4.50          2      7           5
     4.00        4.50                 8           6
     4.00        4.50                 9           7
     0.38        4.50                11           8
     4.00        4.50          3      7           9
     4.00        4.50                 8          10
     4.00        4.50                 9          11
     1.70        4.50                11          12
     4.00        4.50          4      7          13
     4.00        4.50                 8          14
     4.00        4.50                 9          15
     1.73        4.50                11          16
     4.00        4.50          5      7          17
     4.00        4.50                 8          18
     4.00        4.50                 9          19
     1.43        4.50                11          20
     4.00        4.50          6      7          21
     4.00        4.50                 8          22
     4.00        4.50                 9          23
     0.24        4.50                11          24
3        30.0       130.0
24   42   47
25   27      37.34       0.0      47.74    37.340      9.335
32   41      34.75       0.0      69.99    34.710      8.688
39   49      39.17       0.0      46.61    39.170      9.790
45   57      35.00       0.0      85.41    35.000      8.750
50   63      39.17       0.0      46.61    35.000      9.790
57   74      35.00       0.0      69.74    21.400      8.668      0.003
1.93581   2.56606   3.10616   2.75870   2.88290   1.98056
FORT HOOD PUMP OPTIMIZATION DATA   1 AUG 88
 0.000000001          .0001          0.001
 3   15    3    0    0    0    1
```

Figure B1. Sample input file for the main data entry routine.

```
C     ***********************************************************
C     ***********************************************************
C
C       US ARMY COPRS OF ENGINEERS PUMP OPTIMIZATION PROGRAM
C         FOR USE TO DETERMINE LEAST COST PUMP OPERATION
C         GIVEN A VARIABLE ELECTRIC RATE SCHEDULE, SYSTEM
C         DEMANDS, AND PUMP CHARACTERISITICS.
C
C         WRITTEN BY:  DONALD V. CHASE
C                      MARCH/APRIL 1988
C                      US ARMY CORPS OF ENGINEERS
C                      WATERWAYS EXPERIMENT STATION
C                      CEWES-EE-R
C                      P.O. BOX 631
C                      VICKSBURG, MS  39180
C                      601/634-3931
C
C     ***********************************************************
C     ***********************************************************
C
C
C
C     THIS PROGRAM WILL CALL GRG2, A NONLINEAR OPTIMIZATION ROUTINE,
C     AND OPTIMIZE PUMP OPERATION.  DECISION VARIABLES WILL BE TIME
C     OF OPERATION OF EACH PUMP, AND CONSTRAINTS WILL BE WATER LEVEL
C     IN THE SYSTEM TANKS.
C
C
      IMPLICIT REAL*8(A-H,O-Z)
      IMPLICIT INTEGER*4(I-N)

      CHARACTER FILEIN*14,FILEOUT*14,TTITLE*90,ST*90,DATFLE*14
      CHARACTER FILEDT*14,FILEAT*14

      DIMENSION FCNS(150),RMULTS(150),NONBAS(150),REDGR(150),INBIND(150)
      DIMENSION RAMCON(150),RAMVAR(150),DEFAUL(19),XX(150)
      DIMENSION STOMIN(20),Z(20000)
      INTEGER PNL,O

      PARAMETER (PNL=200,LNL=150,MNL=20,IA=2000)
      PARAMETER (NDC=150,NTP=20,NPP=10)

      COMMON /BLK1/ NVAR,NTANK,NSTEP,NNOBJ,C1,NN1,NPUMP,CDEM(100)
      COMMON /BLK2/ ERATE(NDC),AREA(NTP),COEFF(NPP,3),LINK(NPP)
      COMMON /BLK2A/ CTANK(NTP),NNODE(NTP),NCON(NTP),RATIO(24),
     1LFLAG(NPP),NFLAG(NTP)
      COMMON /BLK3/ XO,YO,X1,Y1,X2,Y2
      COMMON /BLK4/ COE1,COE2,COE3
      COMMON /BLK5/ BLVAR(NDC),BUVAR(NDC),BLCON(2*NDC),BUCON(2*NDC)
      COMMON /TANK/ TOPEL(NTP),BOTEL(NTP),ENDEL(NTP),LTANK
      COMMON /NODES/ O(15),HE(PNL),DO(PNL),EL(PNL),S(PNL),G1(PNL)
      COMMON /PIPES/ A(IA),CP(PNL),DI(PNL),XL(PNL),HW(PNL)
      COMMON /TOPOL/ IBE(PNL),IEN(PNL),IPI(PNL),INO(PNL),IBI(PNL),IEI(PN
     1L)
```

Figure B2. Source code for the main data entry routine.

```
      COMMON /PRINT/ IPM,IPP,IPE,NDFLG,IBUG
      COMMON /JOB/ JOB
      COMMON /ACCU/ PRAC,FLAC,HWMA,ICL
      COMMON /FILE/ IIN,IOUT
      COMMON /BLK6/ PSTEP
      COMMON /BLK8/ NNODES,PMIN,PMAX,NNNODE(20),NN2,NN3

      LOGICAL INPRNT,OTPRNT

C
C
C     FILE MANIPULATION BLOCK
C
C
C     WRITE(*,1001)
C     READ(*,1002)FILEIN
C     WRITE(*,1003)
C     READ(*,1002)FILEOUT
C
C
      IIN=5
      IOUT=7
C
C
      OPEN(1IN,ERR=3060,FILE='SYSTEM.DAT',STATUS='OLD',RECL=63)
      OPEN(IOUT,FILE='OPTIMAL.OUT',STATUS='UNKNOWN')

      REWIND 1
      REWIND IIN
      REWIND IOUT

      FILEIN='SYSTEM.DAT'
C
C
C

      GOTO 10
3060  WRITE(IOUT,1004)
      STOP

10    CALL SIMULATE(FILEIN)
      NDFLG=1

C
C     **********************************************************
C     **********************************************************
C
C       ENTER INFORMATION ON DECISION VARIABLES
C
C        DECISION VARIABLES FOR PUMP OPTIMIZATION IS TIME.
C        THE NUMBER OF DECISION VARIABLES IS THE PRODUCT OF THE
C        NUMBER OF PUMPS WHICH CAN BE OPERATED AND THE NUMBER OF
C        TIME STEPS SPECIFIED.  FOR EXAMPLE, IF FIVE (5) PUMPS
C        ARE AVAILALBE TO THE SYSTEM AND THE TIME STEP SPECIFIED
C        IS FOUR (4) HOURS, THE NUMBER OF DECISION VARIABLES IS
```

Figure B2. (Cont'd).

```
C        30.  30 = 5 PUMPS * (24 HOURS/ 4 HOURS)
C
C
C     **********************************************************
C     **********************************************************
C
C     INITIALIZE VALUES FOR GRG ROUTINE
C
C
C     INPRNT:  LOGICAL, ENABLES PRINT OPTION
C               TRUE:  PRINT INPUT DATA
C               FALSE:  DO NOT PRINT INPUT DATA
C
C     OTPRNT:  LOGICAL, ENABLES PRINT OPTION
C               TRUE:  PRINT FINAL RESULTS
C               FALSE:  DO NOT PRINT FINAL RESULTS
C
C     NCORE:   DIMENSION OF THE Z ARRAY
C     NVAR:    NUMBER OF DECISION VARIABLES
C     NFUN:    NUMBER OF FUNCTIONS INCLUDING OBJECTIVE
C     MAXBAS:  UPPER LIMIT ON THE NUMBER OF BINDING CONSTRAINTS
C     MAXHES:  MAXIMUM ALLOWABLE SIZE OF THE HESSION MATRIX
C     NNOBJ:   INDEX OF THE OJECTIVE FUNCTION
C     TTITLE:  ALPHANUMERIC,  TITLE OF PROBLEM
C     BLVAR:   ARRAY OF LOWER BOUND OF VARIABLES
C     BUVAR:   ARRAY OF UPPER BOUND OF VARIABLES
C     BLCON:   ARRAY OF LOWER BOUND OF CONSTRAINTS
C     BUCON:   ARRAY OF UPPER BOUND OF CONSTRAINTS
C     DEFAUL:  ARRAY OF DEFAULT VALUES
C
C     **********************************************************
C     **********************************************************

      LOOP=1

C     WRITE(*,1035)
C     READ(*,1002)FILEDT
C     WRITE(*,1036)
C     READ(*,1002)FILEAT

      OPEN(UNIT=1,FILE='OPTIMAL.DAT',STATUS='OLD')
      OPEN(UNIT=8,FILE='OUT',STATUS='UNKNOWN')


C     NPUMP IS THE NUMBER OF PUMPS AVAIALBE FOR OPERATION

C     NSTEP IS THE TIME STEP SPECIFIED IN HOURS.  A 24 HOUR
C     ANALYSIS IS ASSUMED.

C     LPFLG IS A LOOP FLAG

C     NTANK IS THE NUMBER OF TANKS IN THE SYSTEM CONTRIBUTING TO
C     HEAD SEEN BY THE PUMP STATION
```

Figure B2. (Cont'd).

```
C     PRAC IS THE PRESSURE TOLERANCE USED BY WADISO

C     FLAC IS THE FLOWRATE TOLERANCE USED BY WADISO

C     ICL IS THE ITERATION TOLERANCE USED BY WADISO

C     IBUG IS A PRINT FLAG


      READ(1,1005)NPUMP,NSTEP,LPFLG,NTANK,PRAC,FLAC,ICL,IBUG

      C3=448.831
      FLAC=FLAC/C3
      HWMA=100.


C     NVAR IS THE TOTAL NUMBER OF DECISION VARIABLES
C
C     INTIALIZE CONSTANTS

C     C1 = PUMP COEFFICIENT
C     NN1 = NUMBER OF TIME STEPS
      C1=(62.4*0.746)/550.
      NN1=24/NSTEP

      NVAR=NPUMP*NN1

C     ECHO INPUT
      WRITE(IOUT,1006)NPUMP,NSTEP,NVAR,NTANK
C


C     INTIALIZE ARRAYS
      DO 100 J=1,NPUMP
       LINK(J)=0
100    CONTINUE
        DO  90 J1=1,NVAR
          ERATE(J1)=0.0
90     CONTINUE



C     ENTER PUMP INFORMATION
C      1) LINK NUMBER OF PUMP
C      2) EFFICIENCY INFORMATION
C         a)  FLOW
C         b)  EFF.

      WRITE(IOUT,1007)
      DO 102 I=1,NPUMP
       READ(1,1009)LINK(I),X0,Y0,X1,Y1,X2,Y2
        CALL SCURVE
C
         COEFF(I,1)=COE1
         COEFF(I,2)=COE2
```

Figure B2. (Cont'd).

```
        COEFF(I,3)=COE3
C
       WRITE(IOUT,1008)COEFF(I,1),COEFF(I,2),COEFF(I,3)

       DO 111 I1=1,O(1)
       IF(IPI(I1) .EQ. LINK(I)) THEN
        LFLAG(I)=I1
        GO TO 102
       ENDIF
111    CONTINUE
102   CONTINUE



C     ENTER INITIAL DATA ON DECISION VARIABLES
C
C          1) INITIAL GUESS
C          2) ELECTRICAL RATE
C          3) LOWER BOUND
C          4) UPPER BOUND

      GUESS=0.0
      RLOWBC=0.0
      RUPPBC=0.0
      WRITE(IOUT,1010)
      DO 103 I=1,NVAR
       READ(1,1012) GUESS,ERATE(I)
        XX(I)=GUESS
        BLVAR(I)=0.
        BUVAR(I)=NSTEP
        WRITE(IOUT,1011)I,XX(I),BLVAR(I),BUVAR(I)
103   CONTINUE

C
C      ENTER CONSTRAINT INFORMATION
C
C       1) PRESSURE CONSTRAINT INFORMATION:
C           NNODES - NUMBER OF NODES TO BE CHECK FOR CONSTRAINT VIOLATION
C           PMIN -   MINIMUM ACCEPTABLE PRESSURE (PSI)
C           PMAX -   MAXIMUM ALLOWABLE PRESSUER  (PSI)
C           NNNODE(I) - NODE NUMBER OF NODE TO BE CHECKED
C
C       2) STORAGE TANK CONSTRAINT INFORMATION:
C           NCON -   CONNECTING PIPE NUMBER TO TANK
C           NNODE -  NODE NUMBER OF THE TANK
C           BOTEL -  BOTTOM ELEVATION OF THE TANK
C           TOPEL -  TOP ELEVATION OF THE TANK


      READ(1,1037)NNODES,PMIN,PMAX
      READ(1,1038)(NNNODE(I),I=1,NNODES)
      DO 220 I=1,NNODES
       DO 221 I1=1,O(2)
        IF(INO(I1) .EQ. NNNODE(I))THEN
```

Figure B2. (Cont'd).

```
          NNNODE(I)=I1
          GO TO 220
         ENDIF
221     CONTINUE
220    CONTINUE


        DO 104 I=1,NTANK
         READ(1,1013)NNODE(I),NCON(I),TOPEL(I),BOTEL(I),DIAM,ENDEL(I),
     1   STOMIN(I),PSTEP
         AREA(I)=(3.1415927/4.)*(DIAM*DIAM)
          DO 105 J=1,O(2)
           IF(NNODE(I) .EQ. INO(J)) THEN
            CTANK(I)=DO(J)
            NFLAG(I)=J
           GO TO 104
           ENDIF
105     CONTINUE
104     CONTINUE


       NN2=NN1*NNODES
       NN3=NN1*NTANK
       L44=0

       DO 223 K=1,NN1

         DO 222 I=1,NNODES
           L44=L44+1
           BLCON(L44)=PMIN
           BUCON(L44)=PMAX
222      CONTINUE

        DO 224 I1=1,NTANK
          L44=L44+1
          BLCON(L44)=STOMIN(I1)
          BUCON(L44)=TOPEL(I1)+0.55
C         IF(K .EQ. NN1)BLCON(L44)=ENDEL(I1)
224     CONTINUE

223    CONTINUE


C
C      ENTER INFORMATION ON WATER USE PATTERN.  ENTER RATIO OF WATER USE
C      FOR GLOBAL DEMAND MULTIPLICATION.  ENTER ONE VALUE FOR EACH TIME
C      STEP, NN1

       READ(1,1034)(RATIO(L),L=1,NN1)

C      ENTER NODES OF WATER USE IN AN ARRAY WHICH WILL NOT CHANGE

       L45=0
       DO 99 M1=1,O(2)
        IF(DO(M1) .LE. 0. .OR. DO(M1) .GT. 9E9)GO TO 99
```

Figure B2. (Cont'd).

```
         L45=L45+1
         CDEM(L45)=DO(M1)
99    CONTINUE

C     DETERMINE INTIAL TANK ELEVATIONS

        LTANK=1
        WRITE(IOUT,1014)
        CALL GCOMP(G,XX)
        LTANK=0
        IDON=0

C
C     OTPRNT=.FALSE.
      INPRNT=.TRUE.
      OTPRNT=.TRUE.
      NCORE=20000
      NNVARS=NVAR
      NFUN=NN2+NN3+1
      MAXBAS=NFUN
      MAXHES=NVAR
      NNOBJ=NFUN

C
C
C     READ IN TTITLE
      READ(1,1030)TTITLE
      WRITE(IOUT,1030)TTITLE
C
C
C     ENABLE ALL DEFAULT PARAMETERS
C
      DO 106 K3=1,19
      DEFAUL(K3)=1.0
106   CONTINUE

C     ENABLE INPUT FOR TOLERANCE ON FRACTIONAL CHANGE IN OBJECTIVE
C     FUNCTION (FPSTOP)
C
C     ENABLE INPUT FOR CHANGE IN METHOD OF COMPUTING PARTIAL
C     DERIVATIVES (LDERIV:  0-FOWARD, 1-CENTRAL, 2-USER SUPPLIED)
      DEFAUL(1)=2.0
      DEFAUL(2)=2.0
      DEFAUL(3)=2.0
      DEFAUL(6)=2.0
      DEFAUL(7)=2.0
      DEFAUL(9)=2.0
      DEFAUL(10)=2.0
      DEFAUL(15)=2.0
      DEFAUL(16)=2.0
```

Figure B2. (Cont'd).

```
      DEFAUL(17)=2.0

      READ(1,1031)FPNEWT,FPINIT,FPSTOP
C     SET TOLERENCE PARAMETERS TO DEFAULT VALUES IF READ VALUES OF ZERO

      IF(FPNEWT .EQ. 0.)DEFAUL(1)=1.0
      IF(FPINIT .EQ. 0.)DEFAUL(2)=1.0
      IF(FPSTOP .EQ. 0.)DEFAUL(3)=1.0

      READ(1,1032)NNSTOP,IITLIM,IIPR,IIPN4,IIQUAD,LDERIV,MMODCG
C     SET TOLERENCE PARAMETERS TO DEFAULT VALUES IF READ VALUES OF ZERO

      IF(NNSTOP .EQ. 0.)DEFAUL(6)=1.0
      IF(IITLIM .EQ. 0.)DEFAUL(7)=1.0
      IF(IIPR   .EQ. 0.)DEFAUL(9)=1.0
      IF(IIPN4  .EQ. 0.)DEFAUL(10)=1.0
      IF(IIQUAD .EQ. 0.)DEFAUL(15)=1.0
      IF(LDERIV .EQ. 0.)DEFAUL(16)=1.0
      IF(MMODCG .EQ. 0.)DEFAUL(17)=1.0

C     ECHO VALUES

      WRITE(IOUT,1033)FPNEWT,FPSTOP,NNSTOP,IITLIM,IIPR,IIPN4,IIQUAD,
     1LDERIV,MMODCG

C
C
C     CALL OPTIMIZATION ROUTINE
C

1000  CALL GRGSUB(INPRNT,OTPRNT,NCORE,NNVARS,NFUN,MAXBAS,MAXHES,NNOBJ,
     1TTITLE,BLVAR,BUVAR,BLCON,BUCON,DEFAUL,FPNEWT,FPINIT,FPSTOP,
     2FPSPIV,PPH1EP,NNSTOP,IITLIM,LLMSER,IIPR,IIPN4,IIPN5,IIPN6,IIPER,
     3IIDUMP,IIQUAD,LDERIV,MMODCG,RAMCON,RAMVAR,XX,FCNS,INBIND,
     4RMULTS,NONBAS,REDGR,NBIND,NNONB,INFORM,Z)
C
C
C     DETERMINE FINAL TANK ELEVATIONS
        LTANK=1
        WRITE(IOUT,1014)
        CALL GCOMP(G,XX)

C
      CALL DP
      CALL PRNOUT('C')
C
      DO 998 L3=1,NVAR
998    IF(XX(L3) .LT. 0.25)XX(L3)=0.0
       LTANK=2
       WRITE(IOUT,1014)
      CALL GCOMP(G,XX)
      IBUG=0
      IF(LOOP .LT. LPFLG)GO TO 1000
```

Figure B2. (Cont'd).

```
C     **************  FORMAT BLOCK  **************
C
1001  FORMAT(1X,37(1H*),/,1X,'ENTER THE NAME OF INPUT FILE',/)
1002  FORMAT(A14)
1003  FORMAT(1X,37(1H*),/,1X,'ENTER THE NAME OF OUTPUT FILE',/)
1004  FORMAT(2X,'THE PROGRAM CANNOT ACCESS THIS FILE')
1005  FORMAT(4I5,2F10.2,2I5)
1006  FORMAT(////,2X,'TOTAL NUMBER OF PUMPS OPTIMIZED:',I4,/
     12X,'TIME STEP = ',I5,' HOURS',/
     22X,'TOTAL NUMBER OF DECISION VARIABLES:',I7,/,
     32X,'NUMBER OF TANKS CONTRIBUTING TO HEAD:',I7/)
1007  FORMAT(///,20X,'PUMP EFFICIENCY INFORMATION',//,8X,'COEFFICIENT #1
     1',6X,'COEFFICIENT #2',6X,'COEFFICIENT #3',//)
1008  FORMAT(3F20.8)
1009  FORMAT(I5,6F10.2)
1010  FORMAT(///,28X,'PUMP GROUP INFORMATION',//,2X,'PUMP GROUP',10X,
     1'INITIAL GUESS',10X,'LOWER BOUND',10X,'UPPER BOUND',/)
1011  FORMAT(4X,I3,14X,F10.2,12X,F10.2,11X,F10.2)
1012  FORMAT(2F10.2)
1013  FORMAT(2I5,6F10.2)
1014  FORMAT(///)
1030  FORMAT(A90)
1031  FORMAT(3F15.14)
1032  FORMAT(7I5)
1033  FORMAT(///,2X,65(1H*),/,25X,'OPTIONS DISPLAY',/,2X,'CONSTRAINT TOL
     1ERANCE = ',F15.14,/,2X,'OBJECTIVE FUNCTION TOLERANCE = ',F15.14,/,
     22X,'NUMBER OF OBJECTIVE CHANGE ITERATIONS = ',I5,/,2X,
     3'NUMBER OF NEWTON ITERATIONS = ',I5,/,2X,'PRINT OPTION = ',I5,/,
     42X,'GRADIENT PRINT OPTION = ',I5,/,2X,'BASIC VARIABLE SEARCH = ',
     5I5,/,2X,'METHOD OF OBTAINING PARTIAL DERIVATIVE = ',I5,/,2X,
     6'USE OF CONJUGATE GRADIENT = ',I5,/,2X,65(1H*),//////)
1034  FORMAT(8F10.0)
1035  FORMAT(//,1X,37(1H*),/,1X,'ENTER THE NAME OF SYSTEM DATA FILE',/)
1036  FORMAT(1X,37(1H*),/,1X,'ENTER THE NAME OF INFO DUMP FILE',/)
1037  FORMAT(I5,2F10.2)
1038  FORMAT(16I5)
27    FORMAT(32X,'INITIAL GCOMP CALL')
C
C
      CLOSE(1)
      CLOSE(8)
      CLOSE(IIN)
      CLOSE(IOUT)
999   STOP
      END



      SUBROUTINE SCURVE
C
C
C     THIS SUBROUTINE FITS A QUADRATIC CURVE THROUGH
C     USER SUPPLIED POINTS USING LAGRANGIAN POLYNOMIALS
C
```

Figure B2. (Cont'd).

```
C

      IMPLICIT REAL*8(A-H,O-Z)
      IMPLICIT INTEGER*4(I-N)

      COMMON/BLK3/XO,YO,X1,Y1,X2,Y2
      COMMON/BLK4/COE1,COE2,COE3
      COMMON/FILE/IIN,IOUT
C
C

C
C
      XXO=(XO-X1)*(XO-X2)
      RO=YO/XXO
      XX1=(X1-XO)*(X1-X2)
      R1=Y1/XX1
      XX2=(X2-XO)*(X2-X1)
      R2=Y2/XX2
C
      COE1=RO*X1*X2+R1*XO*X2+R2*XO*X1
      COE2=(-RO*(X1+X2))-(R1*(XO+X2))-(R2*(XO+X1))
      COE3=RO+R1+R2
C
      RETURN
      END
```

Figure B2. (Cont'd).

APPENDIX C:

GCOMP SUBROUTINE

The GCOMP subroutine computes the value of the objective function and problem constraints. This subroutine is required by GRG2. The array containing the most current values of the decision variables, X, is passed to the GCOMP routine for computation of the objective function and constraints. An array containing the computed values of the objective and problem constraints, G, is passed back to GRG2 along with the array containing the current decision variables.

Upon entering the GCOMP routine, values in the X array are transferred to an array named XXX. This transfer switches entries in the one-dimensional X array with NVAR elements to a two-dimensional array also with NVAR elements. The two-dimensional array is an NN1 x NPUMP array where NN1 is equal to the number of time intervals in the 24-hr time period and NPUMP is equal to the number of pumps in the system. NVAR is the number of decision variables included in the optimization.

Essentially, the way the GCOMP routine works is that for each time interval, the pump operating the least amount of time is selected and analyzed. WADISO is called and the system is balanced under the assumption that all pumps with a positive value in the XXX array are operating. If WADISO is unable to balance the network, the coefficient array used by WADISO is recomputed and the network is rebalanced.

After returning to GCOMP, all tanks are evaluated to determine if any tanks that were closed previously should be opened. A tank that is initially closed full is opened when the head at the beginning node of the connecting pipe is less than the head at the ending node (base of the tank) plus the water level in the tank. Conversely, a tank that is initially closed empty will be opened if the head at the beginning of the connecting pipe is greater than the head at the end of the connecting pipe. If a tank is opened, WADISO is called, the system is rebalanced, and the tanks are rechecked until all tanks that should be open are opened.

After all tanks are checked to decide if any should be opened, a check is made to determine if any should be closed. This is accomplished by finding the amount by which the tank drains or fills. A separate subroutine called UPDATE computes the change in water level in all tanks based on the flow rate in tank connecting lines, the area of the tank, and the time of analysis. The time of analysis is equal to the minimum amount of time a pump operates during the time interval evaluated. For example, if three pumps operate during a 4-hr time interval with pump 1 operating for 1 hr, pump 2 operating for 2 hr, and pump 3 operating for 3 hr, the time of analysis would be equal to 1 hr.

If a tank does fill or drain completely, the time to closure is computed. If two or more tanks drain or fill completely, the time to closure is computed for each tank and the minimum time to closure is selected. Tanks are closed by closing the tank connecting line and setting the water elevation in the tank equal to the bottom or top elevation of the tank. The water levels in the remaining tanks are recomputed based on the time to closure if only one tank closes, or the minimum time to closure if multiple tanks close. The time of analysis is then set equal to the minimum time to closure.

The cost of pumping is computed in subroutine COSTC called from GCOMP. At the beginning and end of a time interval, the kilowatt consumption of each pump is determined based on the pump head and pump discharge provided by WADISO, and the pump efficiency. The kilowatt demand for each pump

is averaged and added to find the total average kilowatt demand for the pump station for the time step used. The cost of pumping is found by multiplying the total average kilowatt consumption for the pump station by the electrical rate in effect for this time interval and the time of analysis. The cost of pumping for this small time increment is added to the cumulative cost of pumping thus far to obtain a total cost of pumping for the entire 24-hr time period.

If one or more tanks is closed during the time step analyzed, then the time of analysis is set equal to the minimum time to closure. If this is the case, then the system is rebalanced with those tanks closed as necessary and the entire process of tank analysis and pumping cost analysis is repeated until the end of the time step is reached. Once this point is reached or if no tanks close during the time step, then the entry in the XXX array just analyzed is set equal to zero.

At the end of a time interval, the problem constraints are computed. Problem constraints are the pressure at critical nodes and the water level in each tank. Node pressure is provided by WADISO, whereas tank water level is obtained from the UPDATE subroutine. At this point, the next time interval is analyzed and the entire process repeated until the end of the 24-hr time period is reached.

The source code for the GCOMP routine is shown in Figure C1. The FORTRAN source codes for the UPDATE and COSTC subroutines are shown in Figures C2 and C3.

```
      SUBROUTINE GCOMP(G,X)

      IMPLICIT REAL*8(A-H,O-Z)
      IMPLICIT INTEGER*4(I-N)

      INTEGER PNL,O
      REAL INTLEV

      PARAMETER (PNL=200,LNL=150,MNL=20,IA=2000)
      PARAMETER (NDC=150,NTP=20,NPP=10)

      DIMENSION G(151),X(150),XXX(24,10),NCLOSE(NTP)
      DIMENSION TIME(NTP),TLEV(NTP),TMPLEV(NTP)

      COMMON /BLK1/ NVAR,NTANK,NSTEP,NNOBJ,C1,NN1,NPUMP,CDEM(100)
      COMMON /BLK2/ ERATE(NDC),AREA(NTP),COEFF(NPP,3),LINK(NPP)
      COMMON /BLK2A/ CTANK(NTP),NNODE(NTP),NCON(NTP),RATIO(24),
     1LFLAG(NPP),NFLAG(NTP)
      COMMON /BLK5/ BLVAR(NDC),BUVAR(NDC),BLCON(2*NDC),BUCON(2*NDC)
      COMMON /TANK/ TOPEL(NTP),BOTEL(NTP),ENDEL(NTP),LTANK
      COMMON /NODES/ O(15),HE(PNL),DO(PNL),EL(PNL),S(PNL),G1(PNL)
      COMMON /PIPES/ A(IA),CP(PNL),DI(PNL),XL(PNL),HW(PNL)
      COMMON /TOPOL/ IBE(PNL),IEN(PNL),IPI(PNL),INO(PNL),IBI(PNL),IEI(PN
     1L)
      COMMON /PRINT/ IPM,IPP,IPE,NDFLG,IBUG
      COMMON /JOB/ JOB
      COMMON /FLAG/ R3M,R4M,IFOPP
      COMMON /FILE/ IIN,IOUT
      COMMON /BLK7/ NTNTNT
      COMMON /BLK8/ NNODES,PMIN,PMAX,NNNODE(20),NN2,NN3

C     **************************************************
C
C       LATEST UPDATE ON THIS FILE:   12 DEC 88
C
C     **************************************************



C     THIS IS A SUBROUTINE TO CALL A NETWORK SOLVING ROUTINE TO EVALUATE
C     THE OPERATING CONDITIONS OF A DISTRIBUTION NETWORK


        WRITE(*,*)'  INSIDE GCOMP  '


C     XXX IS A TWO DIMENSIONAL DUMMY ARRAY WITH THE FIRST ELEMENT
C     CORRESPONDING TO THE CURRENT TIME INCREMENT AND THE SECOND
C     ELEMENT CORRESPONDING TO THE PUMP OF INTEREST
C     SWITCH X ARRAY TO XXX ARRY

       L3=0
       IF(NTNTNT .EQ. 1) GO TO 204
       DO 200 L=1,NN1
        DO 201 L1=1,NPUMP
```

Figure C1. Source code for the GCOMP routine.

```
          L3=L3+1
          IF(X(L3) .LE. BUVAR(L3))THEN
           XXX(L,L1)=X(L3)
          ELSE
           HOLD=X(L3)
           XXX(L,L1)=BUVAR(L3)
           IF(L3+NPUMP .LE. NVAR)THEN
            XXX(L+1,L1)=HOLD-XXX(L,L1)+X(L3+NPUMP)
           ELSE
C           XXX(L,L1)=BUVAR(L3)
            XXX(L,L1)=X(L3)
           ENDIF
          ENDIF
       IF(IBUG .GT. 1)WRITE(8,*)'TIME STEP  PUMP NO   DV',L,L1,XXX(L,L1)
201      CONTINUE
200     CONTINUE
        GO TO 205

C       TRANSFER FOR FINITE DIFFERENCE COMPONENT OF GCOMP

204     L3=0
        DO 202 L=1,NN1
         DO 203 L1=1,NPUMP
          L3=L3+1
           XXX(L,L1)=X(L3)
203      CONTINUE
202     CONTINUE


C      REINITIALIZE TANK LEVELS TO SAME LEVEL AT BEGINNING OF DAY
C      OPEN ALL PIPES CONNECTING TANKS AND REINITIALIZE TIME AND
C      TANK LEVEL ARRAYS

205    DO 63 L2=1,NTANK
        J2=NFLAG(L2)
        DO(J2)=CTANK(L2)
        TEMPLEV=(DO(J2)/1E10)-100.
        TIME(L2)=0.0
        TLEV(L2)=0.0
        DO 64 L3=1,O(1)
         IF(IPI(L3) .NE. NCON(L2))GO TO 64
          IF(TEMPLEV .GE. TOPEL(L2))THEN
           CP(L3)=1E20
           NCLOSE(L2)=1
          ELSEIF(TEMPLEV .LE. BOTEL(L2))THEN
           CP(L3)=1E20
           NCLOSE(L2)=2
          ELSE
           CP(L3)=(4.72*XL(L3))/((HW(L3)**1.852)*(DI(L3)**4.87))
           NCLOSE(L2)=0
          ENDIF
64      CONTINUE
63     CONTINUE
```

Figure C1. (Cont'd).

```
C     NTEN IS THE MAXIMUM NUMBER OF WADISO CALLS ALLOWED PER PUMP TIME
C     INCREMENT

      NTEN=NTANK*5
      IF(NTANK .LE. 2)NTEN=20

C     INITIALIZE COST VARIABLES

      TCOST=0.0
      COST=0.0
      L45=0


C     PRIMARY DO LOOP

      DO 100 M=1,NN1


C     INITIALIZE DUMMY AND FLAG VARIABLES

      LLFLG=0
      NTEST=0
      TEST=0.0
      TIME1=0.0
      RTEST=0.0
      NEND=0
      RK1=0.0
      RK2=0.0
      RKSUM=0.0

C     UPDATE DEMAND PATTERN BY MULTIPLYING ALL DEMANDS BY THE GLOABAL
C     DEMAND FACTOR (RATIO)

      L44=0
      IF(RATIO(M) .EQ. 0.)RATIO(M)=1.0
      DO 99 M1=1,O(2)
       IF(DO(M1) .LE. 0. .OR. DO(M1) .GT. 9E9)GO TO 99
        L44=L44+1
        DO(M1)=CDEM(L44)*RATIO(M)
99    CONTINUE


C       FIND PUMP WHICH OPERATES FOR THE SHORTEST DURATION THIS TIME STEP

199   CC=FLOAT(NSTEP)+100000.
      RMIN=CC

      DO 102 I2=1,NPUMP
        IF(XXX(M,I2) .LE. 0.) THEN
         NTEST=NTEST+1
         XL(LFLAG(I2))=-1.0
         GO TO 102
        ELSEIF(XXX(M,I2) .GT. 0. .AND. XXX(M,I2) .LT. RMIN) THEN
         IF(XXX(M,I2) .EQ. TEST) THEN
```

Figure C1. (Cont'd).

```
         XXX(M,I2)=0.0
         NTEST=NTEST+1
         XL(LFLAG(I2))=-1.0
         GO TO 102
        ENDIF
        RMIN=XXX(M,I2)
        MFLAG=I2
        XL(LFLAG(I2))=0.0
       ELSE
        XL(LFLAG(I2))=0.0
       ENDIF
102   CONTINUE

C     ENABLE FLAG LLFLG TO INDICATE NO PUMPS OPERATING THIS TIME STEP

      IF(NTEST .GE. NPUMP) LLFLG=1


C     RESET COUNTER, NCALL, THE NUMBER OF TIMES WADISO IS CALLED
C     PER TIME STEP

      NCALL=0
      TIMESUM=0.0

C     COMPUTE TIME INCREMENT FOR COST COMPUTATION AND TANK LEVEL CHANGE

98    IF(LLFLG .EQ. 1) THEN
       TIME2=NSTEP
      ELSE
       TIME2=XXX(M,MFLAG)
      ENDIF
       DELT=TIME2-TIME1

      CCC=RMIN-TIMESUM
      RMINN=CCC


C     CHECK TO SEE IF NO PUMPS ARE RUNNING (LLFLG=1)

      IF(LLFLG .EQ. 1) THEN

      WRITE(*,2)M
      IF(IBUG .GT. 1)WRITE(8,4)M
      IF(RTEST .GE. NSTEP)GO TO 100

C     ASSURE CONNECTION TO FIXED GRADE

      DO 95 KK=1,NTANK
      IF(IBUG .GT. 1)WRITE(8,*)'TANK NO.   NCLOSE',KK,NCLOSE(KK)

      IF(NCLOSE(KK) .EQ. 0 )GO TO 94
```

Figure C1. (Cont'd).

```
      IF(NCLOSE(KK) .EQ. 1)THEN
       WRITE(*,1007)
       WRITE(*,*)'TANK IS FULL AND NO PUMPS ON:  OPENING TANK LINE'
       NCLOSE(KK)=0
       DO 54 I4=1,O(1)
        IF(IPI(I4) .NE. NCON(KK))GO TO 54
          CP(I4)=(4.72*XL(I4))/(HW(I4)**1.852*DI(I4)**4.87)
54     CONTINUE
      ENDIF
95    CONTINUE

      ELSE
       WRITE(*,1)M,LINK(MFLAG),XXX(M,MFLAG)
       IF(IBUG .GT. 1)WRITE(8,3)M,LINK(MFLAG),XXX(M,MFLAG)
      ENDIF

C     **********************************************************************

94    ZZZ=XXX(M,MFLAG)
      DDD=0.0
      WRITE(*,*)'BALANCE BEFORE TANK LEVELS ADJUSTED'
      CALL BALANCE(NCALL,NTEN,M,ZZZ)
      CALL COSTC(DDD,RK1,RK2,RKSUM)
93    IF(IBUG .GT. 4) CALL PRNOUT('C')

C     **********************************************************************


      DMAX=0.0

C     OPEN TANK LINES

      DO 108 K=1,NTANK
       IF(NCLOSE(K) .EQ. 0)GO TO 108

         DO 111 N1=1,O(1)
          IF(IPI(N1) .NE. NCON(K))GO TO 111

           K1=NFLAG(K)
           IBI1=IBI(N1)
           IEI1=IEI(N1)
           T1=HE(IBI1)
           T2=HE(IEI1)

           IF(NCLOSE(K) .EQ. 1 .AND. T1 .GT. T2)GO TO 108
           IF(NCLOSE(K) .EQ. 2 .AND. T1 .LT. T2)GO TO 108

            HDIFF=ABS(T1-T2)

            IF(HDIFF .GE. DMAX)THEN
             DMAX=HDIFF
             K3=K
             K4=N1
             T11=T1
             T21=T2
```

Figure C1. (Cont'd).

```
            IF(IBUG .GT. 1)WRITE(8,9502)K3,K4,IPI(K4),T11,T21,DMAX
           ELSE
           ENDIF

111       CONTINUE

108    CONTINUE


        IF(IBUG .GT. 1)WRITE(8,9503)K3,NCLOSE(K3),T11,T21,DMAX
        IF(DMAX .EQ. 0.0)GO TO 305
        IF(NCLOSE(K3) .EQ. 1 .AND. T11 .LT. T21 .OR. NCLOSE(K3) .EQ. 2
      1 .AND. T11 .GT. T21) THEN
           CP(K4)=(4.72*XL(K4))/(HW(K4)**1.852*DI(K4)**4.87)
           NCLOSE(K3)=0
           IF(IBUG .LE. 1)GO TO 994
           WRITE(*,*)
           WRITE(*,*)'OPENING TANK NUMBER  ',K3
           WRITE(8,*)'OPENING TANK NUMBER  ',K3
           WRITE(*,*)
994        GO TO 94
          ENDIF


C      FIND THE 'TANK SLOPE' FOR EACH STORAGE TANK

305    IF(LLFLG .EQ. 1) THEN
        T2=NSTEP
       ELSE
        T2=XXX(M,MFLAG)
       ENDIF
       TINC=(T2-TEST)-TIMESUM
       IF(IBUG .LE. 1) GO TO 221
       WRITE(8,*)
       WRITE(8,*)
       WRITE(8,*)'T2          TEST          TIMESUM           TINC'
       WRITE(8,*)T2,TEST,TIMESUM,TINC
       WRITE(8,*)
       WRITE(8,*)
221    DO 106 K=1,NTANK
       CALL UPDATE(TINC,K,INTLEV,WATLEV,TTT)
       TMPLEV(K)=INTLEV
       TLEV(K)=WATLEV
       TIME(K)=TTT
       IF(TIME(K) .LT. 0.0)TIME(K)=0.0
106    CONTINUE


C      FIND THE MINIMUM TIME A TANK VIOLATES BOUNDARY

       DO 107 K=1,NTANK
       IF(IBUG .GT. 1)WRITE(8,*)'TANK  TLEV   TIME',K,TLEV(K),TIME(K)
        IF(NCLOSE(K) .NE. 0)GO TO 107
```

Figure C1. (Cont'd).

```
        IF(TLEV(K) .GE. BOTEL(K) .AND. TLEV(K) .LE. TOPEL(K))GO TO 107
        IF(TIME(K) .LT. RMINN) THEN
         RMINN=TIME(K)
         NHOLD=K
         IF(IBUG .GT. 1)WRITE(8,*)'RMINN   NHOLD',RMINN,NHOLD
        ENDIF
107   CONTINUE

C     TEST TO SEE IF ANY TANK LEVELS WERE VIOLATED (RMINN < CCC)

      IF(RMINN .EQ. CCC)GO TO 105

C     CLOSE THE TANK LINE WHICH VIOLATES A BOUNDARY FIRST

      DO 109 L=1,O(1)
       IF(IPI(L) .NE. NCON(NHOLD)) GO TO 109
        CP(L)=1E20
        GO TO 110
109   CONTINUE

C     ASSIGN HEAD VALUES AT TANK NODES AND ENABLE FLAG INDICATING
C     A CLOSED PIPE.
C     NCLOSE=1, TANK CLOSED FULL
C     NCLOSE=2, TANK CLOSED EMPTY

110   I=NHOLD
      J=NFLAG(I)
      TEMPLEV=TLEV(I)

      IF(TEMPLEV .LE. BOTEL(I))THEN
       WATLEV=BOTEL(I)
       DO(J)=(BOTEL(I)+100)*1E10
       HE(J)=BOTEL(I)+EL(J)
       NCLOSE(I)=2
      WRITE(*,1009)NHOLD,TEMPLEV,BOTEL(I),((M-1)*NSTEP)+RMINN
C     WRITE(8,1009)NHOLD,TEMPLEV,BOTEL(I),((M-1)*NSTEP)+RMINN
      IF(IBUG .GT. 1)WRITE(8,1009)NHOLD,TEMPLEV,BOTEL(I),
     1 ((M-1)*NSTEP)+TEST+RMINN
      ELSEIF(TEMPLEV .GE. TOPEL(I))THEN
       WATLEV=TOPEL(I)
       DO(J)=(TOPEL(I)+100)*1E10
       HE(J)=TOPEL(I)+EL(J)
       NCLOSE(I)=1
      WRITE(*,1010)NHOLD,TEMPLEV,TOPEL(I),((M-1)*NSTEP)+RMINN
C     WRITE(8,1010)NHOLD,TEMPLEV,TOPEL(I),((M-1)*NSTEP)+RMINN
      IF(IBUG .GT. 1) WRITE(8,1010)NHOLD,TEMPLEV,TOPEL(I),
     1 ((M-1)*NSTEP)+TEST+RMINN
      ENDIF


      IF(RMINN .LE. CCC)DELT=RMINN
```

Figure C1. (Cont'd).

```
      IF(TINC .LT. RMINN)DELT=TINC

      DO 103 K=1,NTANK
       J2=NFLAG(K)
       IF(NCLOSE(K) .EQ. 2) THEN
        DO(J2)=(BOTEL(K)+100)*1E10
       ELSEIF(NCLOSE(K) .EQ. 1) THEN
        DO(J2)=(TOPEL(K)+100)*1E10
       ELSE
        DO(J2)=(TMPLEV(K)+100)*1E10
       ENDIF
103    CALL UPDATE(RMINN,K,INTLEV,WATLEV,TTT)
       CALL BALANCE(NCALL,NTEN,M,ZZZ)
       CALL COSTC(RMINN,RK1,RK2,RKSUM)


C     FIND COST OF OPERATING PUMPS FOR TIME=DELT

105   IF(RMINN .LE. CCC)DELT=RMINN
      IF(TINC .LT. RMINN)DELT=TINC

C     TIME ADJUSTMENT

      TIMESUM=TIMESUM+DELT
      IF(IBUG .GT. 1) THEN
      WRITE(8,*)
      WRITE(8,*)'TIME2    TIMESUM',TIME2,TIMESUM
      WRITE(8,*)
      ENDIF
      AABS=ABS(TIME2-TIMESUM)
      IF(AABS .LE. 0.0001) THEN
       TIME1=TIME2
      ELSE
       TIME1=RMINN
      ENDIF


      IF(RMINN .LT. CCC)GO TO 98


      WRITE(*,*)'BALANCE AFTER TANK LEVELS ADJUSTED'
      CALL BALANCE(NCALL,NTEN,M,ZZZ)
      CALL COSTC(DELT,RK1,RK2,RKSUM)

      IF(XXX(M,MFLAG) .EQ. NSTEP .OR. LLFLG .EQ. 1)NEND=1


      IF(NEND .NE. 1)GO TO 332

C     EVALUATE PRESSURE CONSTRAINTS

      DO 331 J=1,NNODES
```

Figure C1. (Cont'd).

```
      L45=L45+1
      J1=NNODE(J)
      G(L45)=(HE(J1)-ABS(EL(J1)))/2.31
      IF(G(L45) .LT. PMIN)WRITE(*,1038)INO(J1),G(L45)
      IF(G(L45) .GT. PMAX)WRITE(*,1039)INO(J1),G(L45)
331   CONTINUE

C     PRINT TANK LEVEL AT EVERY TIME STEP

332    IF(LTANK .GE. 1 )THEN
        IF(LLFLG .EQ. 1)THEN
         WRITE(IOUT,2)M
        ELSE
         WRITE(IOUT,1)M,LINK(MFLAG),XXX(M,MFLAG)
      ENDIF
         WRITE(IOUT,1035)
      ENDIF

      DO 113 I=1,NTANK

      IF(NEND .EQ. 1)THEN
       L45=L45+1
       G(L45)=TLEV(I)
      ENDIF

       IF(LTANK .GE. 1)WRITE(IOUT,1037)I,TLEV(I)
113   CONTINUE

C     COMPUTE COST OF OPERATION THIS TIME INCREMENT

      IF(NEND .NE. 1)GO TO 333
       TCOST=TCOST+(RKSUM*(ERATE(((M-1)*NPUMP)+1))/100.)
       IF(IBUG .GT. 1)WRITE(8,1002)TCOST


333   IF(LTANK .GE. 1)WRITE(IOUT,2000)
      IF(LLFLG .EQ. 1)GO TO 100

      RTEST=XXX(M,MFLAG)
      TIME1=XXX(M,MFLAG)
      XXX(M,MFLAG)=0.

      TEST=RMIN
      NTEST=0

      GO TO 199

100   CONTINUE


      G(NNOBJ)=TCOST
      WRITE(*,1008)TCOST,PENALTY,G(NNOBJ)
      IF(LTANK .EQ. 2)WRITE(IOUT,1008)TCOST,PENALTY,G(NNOBJ)
```

Figure C1. (Cont'd).

```
      IF(IBUG .GT. 1)WRITE(8,1008)TCOST,PENALTY,G(NNOBJ)


      IBUG=1
      IF(IBUG .GT. 3)STOP


C     ***************  FORMAT BLOCK  **************


1     FORMAT(/,2X,'CYCLE NUMBER',I3,10X,'PUMP NUMBER',I5,5X,'TIME THIS P
     1UMP',F10.5)
2     FORMAT(/,2X,'CYCLE NUMBER',I3,10X,'NO PUMPS OPERATING')
3     FORMAT(///,2X,'CYCLE NUMBER',I3,10X,'PUMP NUMBER',I5,5X,'TIME THIS
     1PUMP',F10.5,//)
4     FORMAT(///,2X,'CYCLE NUMBER',I3,10X,'NO PUMPS OPERATING',//)
1001  FORMAT(2X,'********* ERROR ******  DUMMAY ARRAY NOT SWITCHED')
1002  FORMAT(//,2X,'TOTAL COST OF PUMPING THIS TIME INCREMENT:',F10.2)
1004  FORMAT(2X,I10,19X,I10,21X,F10.2)
1005  FORMAT(//,33X,'FLOW INFORMATION',//,4X,'BEGINNING NODE',17X,'ENDIN
     1G NODE',19X,'FLOWRATE',/)
1006  FORMAT(2X,'INITIAL WATER LEVEL:',
     1F10.4,/,2X,'TANK FLOW:',F10.4,/,2X,'FINAL WATER LEVEL:',F10.4,/,
     22X,'TOTAL HEAD AT TANK',F10.4,//)
1007  FORMAT(/,15X,'****  WARNING  ****',/,4X,'NO CONNECTION TO SUPPLY P
     1OINT, MUST TURN ON PUMP',/)
1008  FORMAT(2X,'COST OF PUMPING THIS 24 HR. PERIOD',F10.2,/,
     12X,'PENALTY FUNCTION THIS 24 HR. PERIOD',F10.2,/,
     22X,'OBJECTIVE FUNCTION THIS TIME PERIOD',F10.2,///)
1009  FORMAT(/,20X,'***************   WARNING   ***************',/,2X,
     1'TANK NUMBER',I4,2X,'HAS ELEVATION OF',F7.3,2X,'WHICH VIOLATES LOW
     2ER BOUND CONSTRAINT',F7.3,2X,'OCCURING AT TIME =',F7.4,/)
1010  FORMAT(/,20X,'***************   WARNING   ***************',/,2X,
     1'TANK NUMBER',I4,2X,'HAS ELEVATION OF',F7.3,2X,'WHICH VIOLATES UPP
     2ER BOUND CONSTRAINT',F7.3,2X,'OCCURING AT TIME =',F7.4,/)
1035  FORMAT(/,10X,10(1H*),3X,'FINAL TANK WATER LEVELS',3X,10(1H*),//,
     114X,'TANK NUMBER',20X,'ELEVATION (FT)'/)
1037  FORMAT(15X,I5,24X,F10.3)
1038  FORMAT(/,20X,'***************   WARNING   ***************',/,2X,
     1'NODE NUMBER',I5,3X,'HAS VIOLATED LOWER PRESSURE CONSTRAINT',/,
     22X,'PRESSURE AT THIS NODE = ',F10.2,/)
1039  FORMAT(/,20X,'***************   WARNING   ***************',/,2X,
     1'NODE NUMBER',I5,3X,'HAS VIOLATED UPPER PRESSURE CONSTRAINT',/,
     22X,'PRESSURE AT THIS NODE = ',F10.2,/)
2000  FORMAT(///)
2001  FORMAT(////)
2002  FORMAT(//,2X,'SIMULATION AT TIME =',F10.4,3X,'TANK LINE CLOSED')
2003  FORMAT(2X,'TANK LINE IS NOW OPENED')
3005  FORMAT(/,2X,'HEAD AT BEGINNING NODE',I5,3X,F10.3,/,2X,'HEAD AT END
     1ING NODE',3X,I5,3X,F10.3,/)
9500  FORMAT(//,2X,'TESTING NCLOSE   TANK NUMBER',I5,4X,'NCLOSE',I5,//)
9501  FORMAT(//,2X,'TANK NO.    NCLOSE    B NODE    E NODE    T1  T2',
     14I5,2F10.4)
9502  FORMAT(2X,'K1    K2    LINK NO.    T11    T21     DMAX',3I5,3F10.2)
```

Figure C1. (Cont'd).

```
9503 FORMAT(//,2X,'TANK NO.    NCLOSE     T1   T2   DMAX',2I5,3F10.4)
      RETURN
      END



      SUBROUTINE CHECK
      WRITE(*,*)
      WRITE(*,*)
      WRITE(*,*)'TERMINATING PROGRAM DUE TO INSTABILITIES IN TANK OPERAT
     1ION'
      WRITE(*,*)
      WRITE(*,*)
      WRITE(8,*)
      WRITE(8,*)
      WRITE(8,*)'TERMINATING PROGRAM DUE TO INSTABILITIES IN TANK OPERAT
     1ION'
      WRITE(8,*)
      WRITE(8,*)
      STOP
      RETURN
      END



      SUBROUTINE BALANCE(NCALL,NTEN,M,ZZZ)

      IMPLICIT REAL*8(A-H,O-Z)
      IMPLICIT INTEGER*4(I-N)

      INTEGER PNL,O

      PARAMETER (PNL=200,LNL=150,MNL=20,IA=2000)
      PARAMETER (NDC=150,NTP=20,NPP=10)

      COMMON /BLK2/ ERATE(NDC),AREA(NTP),COEFF(NPP,3),LINK(NPP)
      COMMON /NODES/ O(15),HE(PNL),DO(PNL),EL(PNL),S(PNL),G1(PNL)
      COMMON /PIPES/ A(IA),CP(PNL),DI(PNL),XL(PNL),HW(PNL)
      COMMON /TOPOL/ IBE(PNL),IEN(PNL),IPI(PNL),INO(PNL),IBI(PNL),IEI(PN
     1L)
      COMMON /PRINT/ IPM,IPP,IPE,NDFLG,IBUG
      COMMON /ACCU/ PRAC,FLAC,HWMA,ICL
      COMMON /JOB/ JOB
      COMMON /FLAG/ R3M,R4M,IFOPP
      COMMON /FILE/ IIN,IOUT

      ISUM=0

C     CALL SIMULATION ROUTINE TO BALANCE THE NETWORK
```

Figure C1. (Cont'd).

```
      WRITE(*,*)'CALLING WADISO'
      CALL SIMBAL('O')
      ISUM=ISUM+1

C     CHECK TO SEE IF EXCEED MAXIMUM NUMBER OF CALLS

      NCALL=NCALL+1
      IF(NCALL .GE. NTEN) CALL CHECK


C     CHECK TO ASSURE SYSTEM IS BALANCED, IF NOT RECONSTRUCT SPRASE
C     MATRIX AND REBALANCE.

92    IF(O(9) .EQ. 3) RETURN

C     IFOPP = 100, WADISO FAILURE DUE TO INFINITE LOOP
C     IFOPP = 997, WADISO FAILURE DUE TO DIVISION BY ZERO
C     IFOPP = 998, WADISO FAILURE DUE TO INABILITY TO DELIVER HEAD
C     IFOPP = 999, WADISO FAILURE DUE TO INABILITY TO DELIVER DISCHARGE

      IF(IFOPP .EQ. 996) GO TO 88
      IF(IFOPP .EQ. 997) GO TO 84
      IF(IFOPP .EQ. 998) GO TO 84
      IF(IFOPP .EQ. 999) THEN
       RETURN
      ENDIF
      IF(IFOPP .EQ. 100) GO TO 84

88     WRITE(*,1011)
       IF(IFOPP .NE. 998)O(4)=0

       CALL SIMBAL('O')
       ISUM=ISUM+1
       IF(O(9) .EQ. 3) THEN
        WRITE(*,*)'SUCCESSFUL REBALANCE'
        RETURN
       ENDIF

C       ONE LAST TRY TO REBALANCE

        IF (ISUM .GT. 2)GO TO 90

84      O(4)=0
        IF(IBUG .GT. 1)WRITE(8,*)'ISUM   IFOPP',ISUM,IFOPP
        DO 91 K9K=1,O(1)
         IF(CP(K9K) .GT. 0) THEN
          A(K9K)=1./CP(K9K)
         ELSE
          Y1=CP(K9K)
          Y2=DI(K9K)
          Y3=HW(K9K)
          A(K9K)=1.85/SQRT((Y2*Y2)-2.*Y1*Y3)
         ENDIF
91      CONTINUE
```

Figure C1. (Cont'd).

```
        GO TO 88

90      WRITE(8,1)M,LINK(MFLAG),ZZZ
        ITMP=ICL
        ICL=100
        CALL SIMBAL('O')
        ICL=ITMP
        IF(O(9) .EQ. 3)THEN
        WRITE(*,*)'SUCCESSFUL REBALANCE'
        RETURN
        ENDIF

        WRITE(8,*)
        IF(IBUG .GT. 1)CALL PRNOUT('C')
        WRITE(8,5)R3M/2.308,R4M*448.8
        WRITE(*,5)R3M/2.308,R4M*448.8

C     ******************  FORMAT BLOCK  *******************

1     FORMAT(2X,'CYCLE NUMBER',I3,10X,'PUMP NUMBER',I5,5X,'TIME THIS PUM
     1P',F10.5)
5     FORMAT(//,2X,'MAXIMUM PRESSURE IMBALANCE = ',F6.2,4X,'MAXIMUM FLOW
     1 IMBALANCE = ',F6.2,//)
1011  FORMAT(/,2X,'RECONSTRUCTING SPARSE MATRIX AND REBALANCING',/)


      RETURN
      END
```

Figure C1. (Cont'd).

```
      SUBROUTINE UPDATE(DELT,I,INTLEV,WATLEV,TTT)

      IMPLICIT REAL*8(A-H,O-Z)
      IMPLICIT INTEGER*4(I-N)

      INTEGER PNL,O
      REAL INTLEV

      PARAMETER (PNL=200,LNL=150,MNL=20,IA=2000)
      PARAMETER (NDC=150,NTP=20,NPP=10)

      COMMON /BLK1/ NVAR,NTANK,NSTEP,NNOBJ,C1,NN1,NPUMP,CDEM(100)
      COMMON /BLK2/ ERATE(NDC),AREA(NTP),COEFF(NPP,3),LINK(NPP)
      COMMON /BLK2A/ CTANK(NTP),NNODE(NTP),NCON(NTP),RATIO(24),
     1LFLAG(NPP),NFLAG(NTP)
      COMMON /BLK5/ BLVAR(NDC),BUVAR(NDC),BLCON(2*NTP),BUCON(2*NTP)
      COMMON /TANK/ TOPEL(NTP),BOTEL(NTP),ENDEL,LTANK
      COMMON /NODES/ O(15),HE(PNL),DO(PNL),EL(PNL),S(PNL),G1(PNL)
      COMMON /PIPES/ A(IA),CP(PNL),DI(PNL),XL(PNL),HW(PNL)
      COMMON /TOPOL/ IBE(PNL),IEN(PNL),IPI(PNL),INO(PNL),IBI(PNL),IEI(PN
     1L)
      COMMON /PRINT/ IPM,IPP,IPE,NDFLG,IBUG
      COMMON /JOB/ JOB
      COMMON /FLAG/ IFOPP
      COMMON /FILE/ IIN,IOUT



      J=NFLAG(I)

C     COMPUTE TANK LEVEL FROM WADISO ARRAY DO

      WATLEV=(DO(J)/1E10)-100.

C     FIND THE FLOW IN THE TANK CONNECTING LINE

      DO 106 K=1,O(1)
      IF(IPI(K) .NE. NCON(I))GO TO 106

C     FIND HYDRAULIC GRADE AT BEGINNING AND ENDING NODES

      IBI1=IBI(K)
      IEI1=IEI(K)
      T1=HE(IBI1)
      T2=HE(IEI1)
      DELH=ABS(T1-T2)



C     COMPUTE FLOWRATE

      TFLOW=(DELH/CP(K))**0.54
      FILL=(3600.*TFLOW*DELT)/AREA(I)
      INTLEV=WATLEV
```

Figure C2. Source code for the UPDATE routine.

```
C     DETERMINE DIRECTION OF FLOW, NOTE TANK IS ALWAYS ENDING NODE

      IF(T1 .GT. T2) THEN

C      TANK IS FILLING
       C3=1.
      ELSE

C      TANK IS DRAINING
       C3=-1.
      ENDIF
       GO TO 998
106   CONTINUE

C     UPDATE TANK LEVEL:  WATLEV

998   C4=BOTEL(I)
      C5=TOPEL(I)
      WATLEV=INTLEV+(C3*FILL)
      DO(J)=(WATLEV+100)*1E10
      HEADLEV=WATLEV+EL(J)
      IF(DELT .GT. 0) THEN
       SLOPE=(C3*FILL)/DELT
       IF(SLOPE .GE. 0)THEN
        TTT=(C5-INTLEV)/SLOPE
       ELSE
        TTT=(INTLEV-C4)/ABS(SLOPE)
       ENDIF
      ELSE
       TTT=0.0
      ENDIF


      IF(IBUG .GT. 1. AND. I .LE. 1)WRITE(8,1004)DELT
      IF(IBUG .GT. 1)WRITE(8,1005)I,TFLOW*448.84,INTLEV,(C3*FILL),
     1 WATLEV,HEADLEV


C     ***************  FORMAT BLOCK  **************

1001  FORMAT(2I6,F13.4,I6,6F10.4)
1002  FORMAT(///,' TANK NO.    B NODE        HEAD              E NODE      HEAD
     1',/)
1003  FORMAT(I6,I10,F14.4,6X,I6,F14.4)
1004  FORMAT(/,5X,'TIME STEP THIS PERIOD',F10.4,
     1//,' TANK NO.   FLOWRATE      INTLEV        DEL H         WATLEV
     2 HEAD AT TANK',/)
1005  FORMAT(I6,5F13.4)
      RETURN
      END
```

Figure C2. (Cont'd).

```
      SUBROUTINE COSTC(DELT,RK1,RK2,RKSUM)

      IMPLICIT REAL*8(A-H,O-Z)
      IMPLICIT INTEGER*4(I-N)

      INTEGER PNL,O

      PARAMETER (PNL=200,LNL=150,MNL=20,IA=2000)
      PARAMETER (NDC=150,NTP=20,NPP=10)

      COMMON /BLK1/ NVAR,NTANK,NSTEP,NNOBJ,C1,NN1,NPUMP,CDEM(100)
      COMMON /BLK2/ ERATE(NDC),AREA(NTP),COEFF(NPP,3),LINK(NPP)
      COMMON /BLK2A/ CTANK(NTP),NNODE(NTP),NCON(NTP),RATIO(24),
     1LFLAG(NPP),NFLAG(NTP)
      COMMON /BLK5/ BLVAR(NDC),BUVAR(NDC),BLCON(2*NTP),BUCON(2*NTP)
      COMMON /TANK/ TOPEL(NTP),BOTEL(NTP),ENDEL,LTANK
      COMMON /NODES/ O(15),HE(PNL),DO(PNL),EL(PNL),S(PNL),G1(PNL)
      COMMON /PIPES/ A(IA),CP(PNL),DI(PNL),XL(PNL),HW(PNL)
      COMMON /TOPOL/ IBE(PNL),IEN(PNL),IPI(PNL),INO(PNL),IBI(PNL),IEI(PN
     1L)
      COMMON /PRINT/ IPM,IPP,IPE,NDFLG,IBUG
      COMMON /JOB/ JOB
      COMMON /FLAG/ IFOPP
      COMMON /FILE/ IIN,IOUT


      IF(RK2 .LE. 0.0)GO TO 9998
       RK1=RK2



C      FIND FLOW AND HEAD ASSOCIATED WITH EACH PUMP THAT IS OPERATING

9998   IF(IBUG .GT. 1)WRITE(8,1012)
       RKW=0.0
        DO 103 I=1,NPUMP
         J1=LFLAG(I)
           IF(A(J1) .EQ. 0.) THEN
            QADD=0.
           ELSE
            Z1=CP(J1)
            Z2=DI(J1)
            QPCFS=(-Z2-1.85/A(J1))/2/Z1
            QPGPM=QPCFS*448.8
            EFFI=COEFF(I,1)+(COEFF(I,2)*QPGPM)+(COEFF(I,3)*QPGPM*QPGPM)
            QADD=QPCFS/(EFFI/100.)
           ENDIF


C          FIND PUMP HEAD

           H1=HE(IBI(J1))
           H2=HE(IEI(J1))
           HEAD=ABS(H1-H2)
```

Figure C3. Source code for COSTC routine.

```
C      COMPUTE KILOWATT DEMAND FOR EACH PUMP AND TOTAL PUMP STATION
C      KILOWATT DEMAND

       ZKW=C1*HEAD*QADD
       RKW=RKW+ZKW
       IF(IBUG .GT. 1 .AND. ZKW .GT. 0.)WRITE(8,1011)LINK(I),HEAD,
     1 QPGPM,EFFI,ZKW

103    CONTINUE

       RK2=RKW
       IF(RK1 .LE. 0.0 .AND. DELT .LE. 0.0)RKAREA=0.0
       IF(RK1 .LE. 0.0)GO TO 9999
       RKAREA=(DELT*(RK1+RK2))/2.
       RKSUM=RKSUM+RKAREA

9999   IF(IBUG .GT. 1)WRITE(8,1002)
       IF(IBUG .GT. 1)WRITE(8,1003)HEAD,RK1,RK2,DELT,RKAREA
       IF(IBUG .GT. 1)WRITE(8,1004)RKSUM

C     ***************  FORMAT BLOCK  **************

1002   FORMAT(//,33X,'PUMP INFORMATION',//9X,'HEAD',9X,'KW #1 USED',4X,
     1'KW #2 USED',7X,'DELTA T',7X,'AVG. KW USED',/)
1003  FORMAT(5F15.4)
1004  FORMAT(/,5X,'ACCUMULATED KILOWATT USAGE:',F10.4)
1011  FORMAT(2X,I5,11X,F10.2,10X,F10.2,5X,F10.2,4X,F10.2)
1012  FORMAT(//,2X,'PUMP NUMBER',8X,'PUMP HEAD',7X,'PUMP DISCHARGE',
     15X,'EFFICIENCY',6X,'KW',//)


       RETURN
       END
```

Figure C3. (Cont'd).

APPENDIX D:

FORT HOOD HYDRAULIC NETWORK DATA

Table D1

Pipe Data for Fort Hood Main Post

Pipe Number	Connecting Nodes	Diameter (in.)	Length (ft)	Hazen-Williams C-Factor	
1	1	2	30	200	100
2	2	3	30	1	100
3	2	5	30	1	100
4	2	7	30	1	100
5	2	9	30	1	100
6	2	11	30	1	100
7	3	4	Pump	-	-
8	5	6	Pump	-	-
9	7	8	Pump	-	-
10	9	10	Pump	-	-
11	11	12	Pump	-	-
12	4	13	30	1	100
13	6	13	30	1	100
14	8	13	30	1	100
15	10	13	30	1	100
16	12	13	30	1	100
20	13	15	30	200	100
21	15	16	24	120	100
22	16	18	24	200	100
23	15	17	30	160	100
24	17	18	30	195	100
25	16	19	12	2800	35
26	19	20	12	1020	35
27	20	25	12	100	100
28	20	21	8	1350	35
29	19	70	8	880	35
30	21	22	8	3220	35
31	22	23	12	1500	35
32	23	90	12	5060	35
33	18	24	24	2520	84
34	24	65	24	1800	84
35	18	26	16	210	69
36	26	27	8	3860	100
37	26	28	16	2080	69
38	28	72	16	900	69
39	29	31	16	3080	69
41	31	32	16	50	100
42	33	36	8	9733	100
43	31	34	20	9420	96
44	34	35	20	3820	96
46	37	75	24	3100	84
47	37	40	18	1020	102
48	36	74	12	2130	102
49	38	39	12	135	100

Table D1 (Cont'd)

Pipe Number	Connecting Nodes		Diameter (in.)	Length (ft)	Hazen-Williams C-Factor
50	38	40	16	520	102
51	40	42	12	4360	102
52	37	76	18	2275	112
53	41	42	10	1600	112
54	41	43	16	1840	112
55	43	46	12	2280	130
56	17	44	20	9372	130
57	44	45	20	60	100
58	44	46	12	2070	130
59	46	79	10	1960	130
60	47	48	10	1160	130
61	43	78	16	2090	130
62	48	49	10	3130	130
63	49	50	18	80	100
64	47	51	8	3640	103
65	49	52	18	540	130
66	52	53	12	1620	130
67	53	54	8	1020	130
68	41	53	8	2440	130
69	42	54	8	2480	130
70	53	55	12	5020	130
71	54	55	8	6230	130
72	51	56	8	900	103
73	49	56	12	7650	130
74	56	57	12	4501	00
75	56	58	12	2550	130
76	58	80	8	1560	130
77	55	59	10	3920	130
78	59	60	10	2820	130
79	58	61	10	4060	130
80	60	62	8	2960	130
81	61	63	8	2290	130
82	33	34	8	3330	100
83	21	70	8	440	35
84	65	75	24	70	84
85	38	74	12	1880	102
86	29	72	16	920	69
87	41	76	18	1550	112
88	52	78	16	962	130
89	47	79	10	540	130
90	59	80	8	1090	130
91	62	81	8	880	130
92	63	81	8	1000	130
100	24	90	PRV AT 85.0 psi		
101	16	91	PRV AT 107.0 psi		

Table D2

Node Data for Fort Hood Main Post

Node Number	Elevation (ft)	Node Demand (gpm)
1	860.0	Reservoir
2	853.0	0
3	853.0	0
4	853.0	0
5	853.0	0
6	853.0	0
7	853.0	0
8	853.0	0
9	853.0	0
10	853.0	0
11	853.0	0
12	853.0	0
13	853.0	0
15	853.0	0
16	854.0	0
17	853.0	0
18	859.0	0
19	864.0	29
20	906.0	42
21	876.0	16
22	877.0	25
23	857.0	31
24	877.0	120
25	1049.0	Tank
26	865.0	0
28	902.0	41
29	910.0	73
30	911.0	0
31	918.0	100
32	1051.6	Tank
33	892.0	96
34	918.0	200
36	915.5	90
37	940.0	0
38	945.0	0
39	1047.2	Tank
40	953.0	150
41	936.0	200
42	953.0	110
43	911.0	500
44	950.0	0
45	1051.3	Tank

Table D2 (Cont'd)

Node Number	Elevation (ft)	Node Demand (gpm)
46	912.0	113
47	968.0	106
48	955.0	26
49	946.0	111
50	1047.2	Tank
51	952.0	0
52	941.0	88
53	941.0	36
54	943.0	291
55	898.0	58
56	929.0	117
57	1051.3	Tank
58	909.0	96
59	899.0	54
60	847.0	33
61	874.0	64
62	870.0	45
63	926.0	52
65	877.0	65
70	877.0	0
72	906.0	0
74	924.0	0
75	912.0	0
76	937.0	0
78	930.0	0
79	965.0	0
80	888.0	0
81	923.0	0
90	877.0	0
91	854.0	0

USACERL DISTRIBUTION

Chief of Engineers
ATTN: CEHEC-IM-LH (2)
ATTN: CEHEC-IM-LP (2)
ATTN: CEMP-EB
ATTN: CECC-P
ATTN: CECW
ATTN: CECW-O
ATTN: CECW-P
ATTN: CECW-RR
ATTN: CEMP
ATTN: CEMP-C
ATTN: CEMP-E
ATTN: CERD
ATTN: CERD-L
ATTN: CERD-C
ATTN: CERD-M
ATTN: CERM
ATTN: DAEN-ZCE
ATTN: DAEN-ZCI
ATTN: DAEN-ZCM
ATTN: DAEN-ZCZ

CEHSC
ATTN: CEHSC-ZC 22060
ATTN: DET III 79906
ATTN: CEHSC-F 22060
ATTN: CEHSC-TT-F 22060

US Army Engineer Districts
ATTN: Library (40)

US Army Engr Divisions
ATTN: Library (14)

US Army Europe
ODCS/Engineer 09403
ATTN: AEAEN-FE
ATTN: AEAEN-ODCS
V Corps
ATTN: DEH (11)
VII Corps
ATTN: DEH (16)
21st Support Command
ATTN: DEH (12)
USA Berlin
ATTN: DEH (9)
Allied Command Europe (ACE)
ATTN: ACSGEB 09011
ATTN: SHIHB/Engineer 09055
ATTN: AEUES 09168
USASETAF
ATTN: AESE-EN-D 09019

8th USA, Korea (19)

ROK/US Combined Forces Command 96301
ATTN: EUSA-HHC-CFC/Engr

USA Japan (USARJ)
ATTN: DCSEN 96343
ATTN: Facilities Engineer 96343
ATTN: DEH-Okinawa 96331

Area Engineer, AEDC-Area Office
Arnold Air Force Station, TN 37389

416th Engineer Command 60623
ATTN: Facilities Engineer

US Military Academy 10996
ATTN: Facilities Engineer
ATTN: Dept of Geography &
Computer Sciences
ATTN: MAEN-A

AMC - Dir., Inst., & Svcs.
ATTN: DEH (22)

DLA ATTN: DLA-WI 22304

DNA ATTN: NADS 20305

FORSCOM (28)
FORSCOM Engineer, ATTN: Spt Det. 15071
ATTN: DEH

HSC
Ft. Sam Houston AMC 78234
ATTN: HSLO-F
Fitzsimons AMC 80045
ATTN: HSHG-DEH
Walter Reed AMC 20307
ATTN: Facilities Engineer

NARADCOM, ATTN: DRDNA-F 01760

TARCOM, Fac, Div. 48090

TRADOC (19)
HQ, TRADOC, ATTN: ATEN-DEH 23651
ATTN: DEH

TSARCOM, ATTN: STSAS-F 63120

USAIS
Fort Huachuca 85613
ATTN: Facilities Engineer (3)
Fort Ritchie 21719

WESTCOM
Fort Shafter 96858
ATTN: DEH
ATTN: APEN-A

SHAPE 09055
ATTN: Survivability Sect. CCB-OPS
ATTN: Infrastructure Branch, LANDA

HQ USEUCOM 09128
ATTN: ECJ 4/7-LOE

Fort Belvoir, VA
ATTN: Australian Liaison Officer 22060
ATTN: Water Resource Center 22060
ATTN: Engr Studies Center 22060
ATTN: Engr Topographic Lab 22060
ATTN: ATZA-TE-SW 22060
ATTN: CECC-R 22060

CECRL, ATTN: Library 03755

HQ, XVIII Airborne Corps and
Ft. Bragg 28307
ATTN: AFZA-DEH-EE

Chanute AFB, IL 61868
3345 CES/DE, Stop 27

NAVFAC
ATTN: Division Offices (11)
ATTN: Facilities Engr Cmd (9)
ATTN: Naval Public Works Center (9)
ATTN: Naval Civil Engr Lab (3)
ATTN: Naval Constr Battalion Ctr 93043

Engineering Societies Library
New York, NY 10017

National Guard Bureau 20310
Installation Division

US Government Printing Office 20401
Receiving/Depository Section (2)

US Army Env. Hygiene Agency
ATTN: HSHB-ME 21010

Nat'l Institute of Standards & Tech 20899

Defense Technical Info. Center 22304
ATTN: DTIC-FAB (2)

298
8/90